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THE UNIVERSITY OF ALBERTA

THREE-DIMENSIONAL ANALYSIS OF MICA DAM

by

Ⓒ JOHN VINCENT SIMMONS

A THESIS
SUBMITTED TO THE FACULTY OF GRADUATE STUDIES AND RESEARCH
IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE
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DEPARTMENT OF CIVIL ENGINEERING

EDMONTON, ALBERTA
FALL, 1974

THE UNIVERSITY OF ALBERTA

FACULTY OF GRADUATE STUDIES AND RESEARCH

The undersigned certify that they have read, and recommend to the Faculty of Graduate Studies and Research, for acceptance, a thesis entitled "Three-Dimensional Analysis of Mica Dam," submitted by John Vincent Simmons in partial fulfilment of the requirements for the degree of Master of Science.

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Date . . 29 April 1974 . .

FRONTISPIECE: MICA DAM, MAY 1973

ABSTRACT

Three-dimensional finite element analyses were performed to investigate the stresses and settlements during construction of Mica Dam. This high earthfill structure in a skewed steep valley was a suitable subject for studies to develop further confidence in simplified analytical approaches which can successfully model complex field situations.

Despite the economically necessary coarseness of the analyses, representative behaviour matching field measurements and tests was calculated using a simple linear elastic approach. Conclusions are drawn as to the situations where such a simplified representation of soil behaviour is valid.

Confirmation of previous investigations was made, suggesting that the stress state in the core of the dam is very much less than expected. Some implications of this important feature are discussed.

A series of parametric studies of a homogeneous Mica Dam showed that a Young's modulus of 7800 Ksf. and a Poisson's ratio of 0.28 yielded good estimates of shell settlements. Similar values for core settlements were 4500 Ksf and 0.35.

Analyses of the zoned embankment were made using simple linear elastic parameters, including bedrock settlements, and also a 2D plane strain condition was imposed. A simple approach to nonlinear behaviour was made by allowing deformation moduli to vary with stress level. Average valuesof Young's modulus and Poisson's ratio were 7800 Ksf and 0.28 for the shell, and about 2400 Ksf and 0.33 for the core, to match field settlements.

It was demonstrated that the dominant deformational behaviour of Mica Dam during construction could be satisfactorily represented by 2D plane strain analyses. Stress distributions for various analyses are given to emphasize the low stress state in the core.

ACKNOWLEDGEMENTS

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The author wishes to thank all of his friends who contributed so much to the state of mind required to see the work through, and especially thanks Mr. T. C. Law for

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All field data was made available by permission of the British Columbia Hydro Power Authority, through the auspices of CASECO Consultants, Vancouver. Mr. N. A. Skermer, of CASECO, is particularly thanked for his guidance, criticisms, and suggestions during this research.

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CHAPTER I

INTRODUCTION: SCOPE OF THIS STUDY

1.1 Role of Deformation Studies in Design and Monitoring of Embankments

> As part of the design process the engineer must estimate the movements, strains, and pore water pressures which may develop in his dam at various times in its life. He makes these estimates largely on the basis of laboratory tests and theoretical analyses, but since the theories are still in a state of development he can only rely on them to the extent that they have been demonstrated to check actual measurements on completed structures.

Sherard, et al. Earth and Earth-Rock Dams, 1963.

Once estimates are made it is the engineers' responsibility to ensure that the structure exhibits the behaviour predicted. If deviations from predictions are significant the questions "why" must be answered, particularly in the context of safety of the structure.

Significant advances in analytical power have become available with finite element techniques, particularly in dealing with stresses, deformations, seepage, and consolidation. The wealth of information potentially available at the design stage of a project is staggering when the whole rather than the parts can be observed, namely with 3D analyses. The engineer's perception of his ability to know "what is going on" is being expanded greatly and personal adjustments need to be made to this new era.

Instrumentation programs have been widely used for some time now, but it is only in the age of the finite element where the full potential of behaviour monitoring can really be appreciated. The new analytical power is yielding far more than before so there is even more reason to check field behaviour to verify analyses. The requirements and responsibilities demanded by society's new and growing environmental consciousness accentuate the need to know in detail "what is going on" with a project and make this knowledge freely available. This thesis aims at being a contribution to such need.

In this chapter background concepts for the thesis are discussed. Succeeding chapters amplify the details and present the results, and conclusions are made specifically regarding this study on Mica Dam but more generally about the role this work is designed to fulfil.

1.2 Importance of Documented Case Histories

The economy of earth and rockfill structures lies in the utilization of readily available materials, and their performance is a function of the engineering efforts put into selection, design, and construction. Usually one cannot transfer data from site to site since each project is essentially unique. An analysis manipulates data from one site and in the process many assumptions, usually whole series of assumptions, are made.

To verify that the analysis is valid measurements are made at the site, and the agreements made and questions raised remain attached to that site. To transfer these experiences of results, measurements, comparisons and questions it is necessary to document the whole situation thoroughly. Only by doing this can confidence be born in the versatility of the analytical procedures. This process is accentuated by a continuing stream of theoretical and analytical developments. At the level where most engineers work numerical results are required. When an analysis produces these results, and documentation is there to support it, the general well-being of the discipline is advanced.

1.3 Problems of Arching and Stress Transfer

The overburden stress at a point in a soil mass is equal to the weight of materials in a vertical column of unit area above the point. It is independent of any stress-strain parameters in an ideal, semi-infinite continuum, and is also the maximum principal stress,

$$\sigma_1 = \Sigma\gamma h$$

where σ_1 = maximum principal stress at a point

γ = material unit weight

h = vertical extent of the material above the point in consideration

To obtain the other two principal stresses involves stress-strain theory, and if linear elasticity is assumed then

$$\sigma_2 = \sigma_3 = K_o \sigma_1$$

where $K_o = \frac{\mu}{1-\mu}$ (a special case really)

μ = Poisson's Ratio

σ_2, σ_3 = intermediate, minor principal stresses respectively.

If the stress distribution in a soil mass is altered from this simple state by an imposed boundary condition, then in general the major principal stress will not equal the overburden stress and in particular it will usually be less than overburden. The difference may be considered as taken up in stress redistribution to satisfy equilibrium at the boundaries. The phenomenon of stress changes to satisfy equilibrium and compatability under boundary influences is loosely termed stress transfer, load transfer, or arching (since the principles are embodied in structural arches).

In embankment engineering it is common to talk of "cross valley arching" since the process so closely parallels the distribution of stresses in an arch bridge across a valley. Furthermore if the embankment consists of two different materials and a softer material "hangs up" on a stronger one major principal stresses in the two decrease and increase respectively from the stress state present if they had identical properties, and we speak of "load transfer" or "stress transfer". Since it is useful to separate the effects if possible, this thesis will consistently use the term "arching" to mean cross-valley stress transfer due to geometrical configuration of the boundaries of an embankment across a valley. "Stress transfer" will refer to the effects of variations in deformation behaviour, and will also be used where the geometrical and deformational effects cannot be separated.

In three dimensions the stress state at a point is quantitatively represented by the octahedral normal and shear stresses,

$$\sigma_{oct} = \frac{1}{3}(\sigma_1 + \sigma_2 + \sigma_3)$$

$$\tau_{oct} = \frac{1}{3}\{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\}^{\frac{1}{2}}$$

If no stress transfer effects are present and all stresses are calculated using simple elastic theory (with only two independent parameters), then the same elastic theory yields

$$\frac{\sigma_{oct}}{\sigma_1} = \frac{1}{3}\ \frac{(1 + \mu)}{(1 - \mu)}$$

and any deviations from this ratio thus indicate the presence of boundary effects or differences in deformational behaviour. This is an interesting idea but becomes rather impractical, consequently any evaluation of stress transfer or arching is made in this thesis by considering the relative values of overburden stress and major principal stress. In most situations where stresses are due to material self-weight only, the major principal stress is close in magnitude and often in direction to the vertical stress.

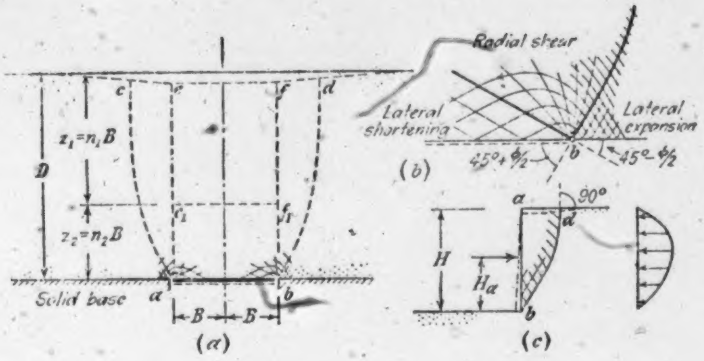
1.4 Review of Some Stress Transfer Effects in Soils

In practically all engineering situations stress transfer effects will be present. It is necessary to have a feel for the processes and effects since stress transfer can have important implications.

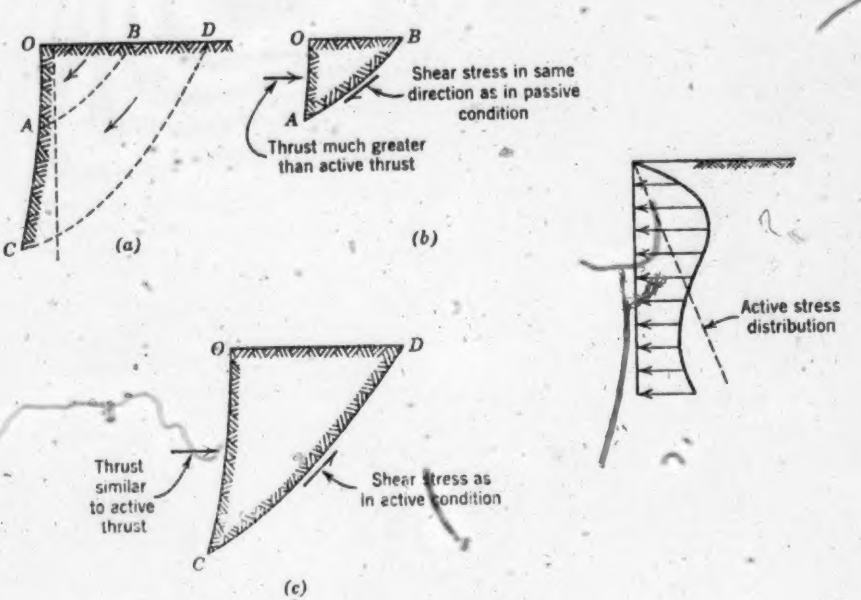
Terzaghi (1943) devotes discussion to the simplest example of stress transfer, arching in this context. He clearly develops the concept that shear stresses are mobilized in order to satisfy equilibrium in a mass when boundary conditions are modified. Similarly, concepts useful in the understanding and design of braced excavations can be developed. (Terzaghi and Peck, 1967; Lambe and Whitman, 1969). Refer to Fig. 1.1.

Several examples of arching and stress transfer are cited by Tschebotarioff (1973) in the context of consolidating fills, vibrations, and flexible bulkheads. He indicates clearly how the mobilization of shear stress in response to boundary conditions influences the overall stress distribution on the structures discussed.

A theoretical study of the development of stresses over flexible and rigid culverts within a soil mass, including


Radial shear
c
e
f
d
Lateral shortening
Lateral expansion
$z_1 = n_1 B$
D
$z_2 = n_2 B$
e_1
f_1
Solid base
a
b
B
B
(a)
$45° + \phi/2$
b
$45° - \phi/2$
(b)
a
90°
d
H
H_a
b
(c)

Modifying boundaries (after Terzaghi, 1943)


O
B
D
A
C
(a)
O
B
A
Shear stress in same direction as in passive condition
Thrust much greater than active thrust
(b)
Active stress distribution
O
D
Thrust similar to active thrust
Shear stress as in active condition
C
(c)

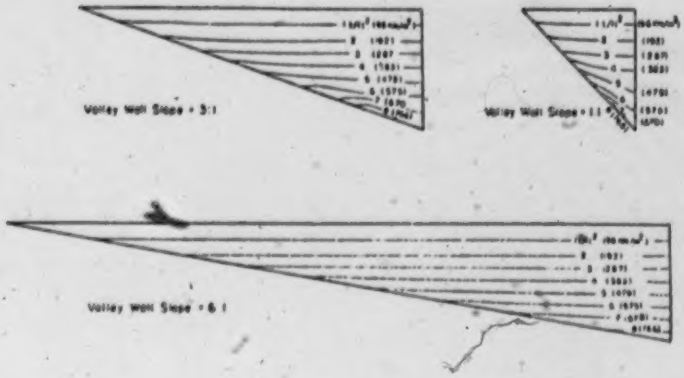
Braced excavations (after Lambe and Whitman, 1969)

FIGURE 1.1 STYLISED DEVELOPMENT OF STRESS TRANSFER

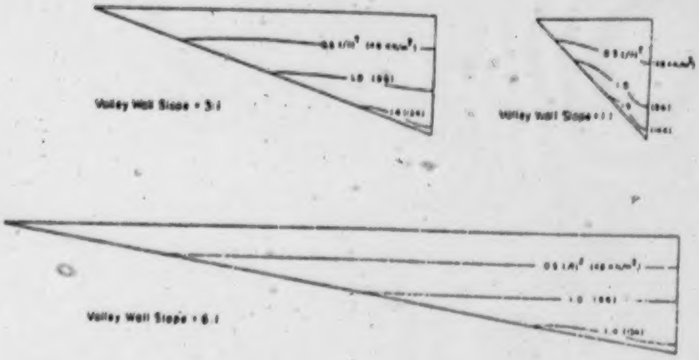
the effects of different construction methods, was reported by Costes (1956). Central to his analysis is the satisfying of equilibrium between different portions of the soil mass/ culvert system by development of shear stresses along boundary surfaces. Some rather arbitrary estimates of arching effects are also involved, and he justifies these on the basis of field observations.

Stress transfer is always present in some form in embankments, particularly zoned fills. Kulhawy and Duncan (1970) performed plane strain finite element analyses of Oroville Dam, and indicate that the analysis explains quite well the construction behaviour of the structure. The stress transfer between core and stiffer shells, and around the concrete core block, were clearly shown. By plotting the shear stresses in terms of strength mobilized toward failure (using the Mohr-Coulomb criterion) they demonstrated the relatively high proportions of strength mobilized at the sites of stress transfer processes.

Lefebvre, Duncan and Wilson (1973) compared 2D and 3D analyses of an idealized homogeneous earth dam, to illustrate some features of finite element analysis of embankments. Arching is clearly shown by the effects of valley slope steepness (Fig. 1.2). In a similar study Palmerton and Lefebvre (1972) demonstrated the strength mobilization against the valley walls accompanying arching, and similarly demonstrated stress transfer between shells and core of

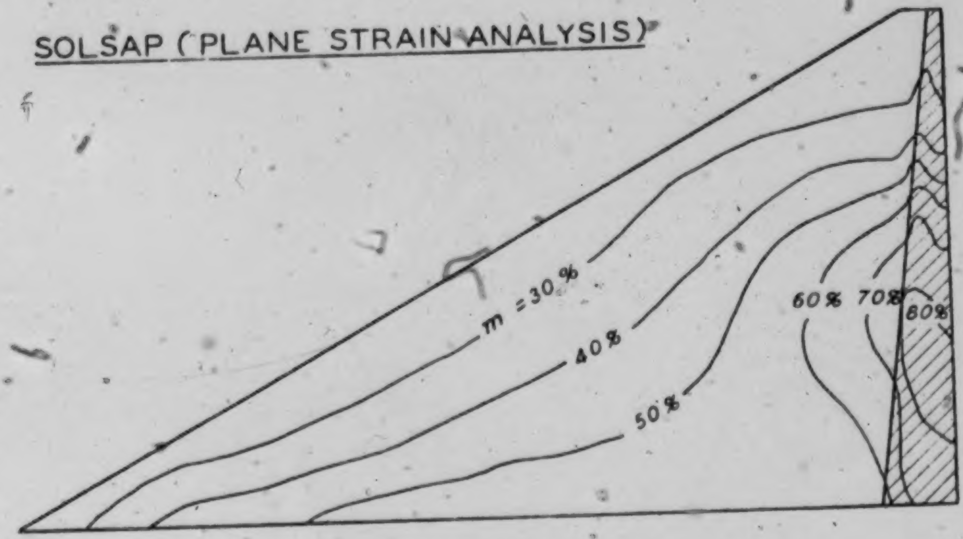

Valley Wall Slope = 3:1
Valley Wall Slope = 1:1
Valley Wall Slope = 6:1

Major Principal Stress Contours, Longitudinal Section


Valley Wall Slope = 3:1
Valley Wall Slope = 1:1
Valley Wall Slope = 6:1

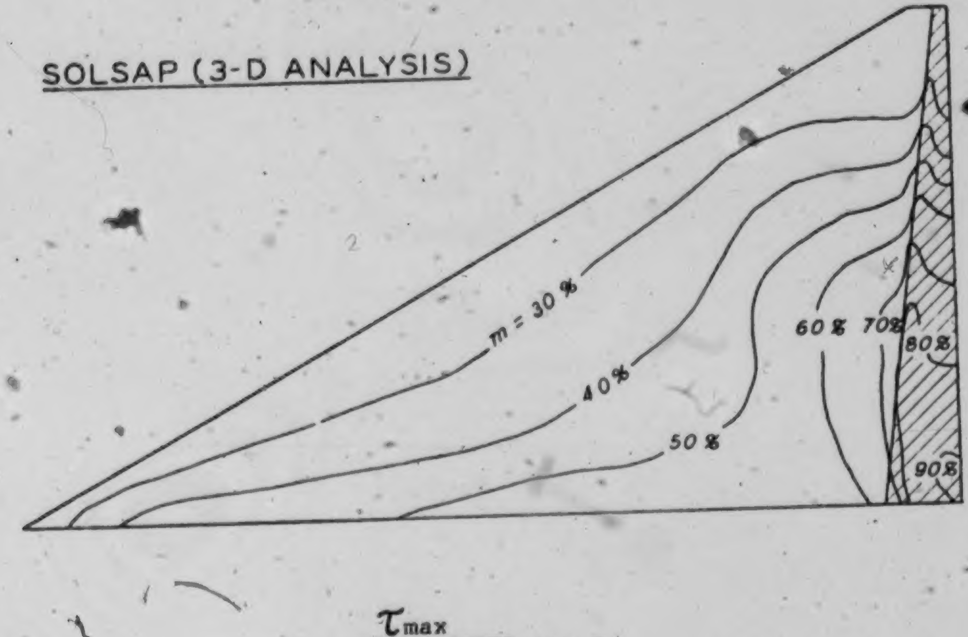
Maximum Shear Stress Contours, Longitudinal Section

FIGURE 1.2 EFFECT OF VALLEY STEEPNESS ON ARCHING
(after Lefebvre, Duncan, and Wilson, 1973)



SOLSAP (PLANE STRAIN ANALYSIS)

m =30%
40%
50%
60%
70%
80%



SOLSAP (3-D ANALYSIS)

m = 30%
40%
50%
60%
70%
80%
90%

$$m = \frac{\tau_{max}}{\tau_{failure}} \times 100\ \%$$

FIGURE 1.3 STRENGTH MOBILIZATION DUE TO CORE-SHELL STRESS TRANSFER (after Palmerton and Lefebvre, 1972)

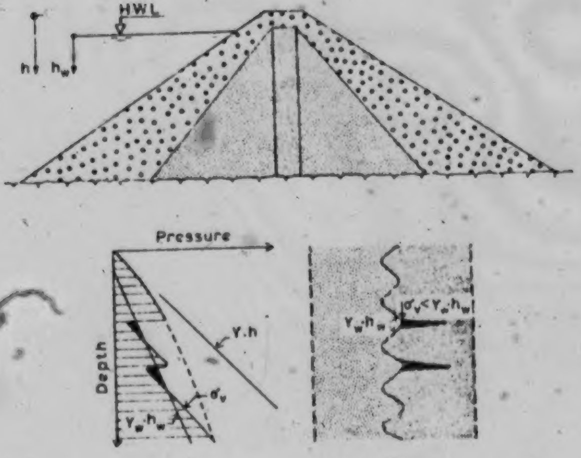

HWL
h
h_w
Pressure
Depth
$\gamma \cdot h$
σ_v
$\gamma_w \cdot h_w$
$\gamma_w \cdot h_w$
$\sigma_v < \gamma_w \cdot h_w$

FIGURE 1.4 HYDRAULIC FRACTURE MECHANISM
(after Kjaernsli and Torblaa, 1968)

different properties (Fig. 1.3).

Because of the stress transfer and arching effects, the stresses in the core of an earth dam are usually less than overburden. Schober (1970) presented a rigorous study of the stress distributions in Gepatsch Dam, where field-measured earth pressures in the core were at least 50% less than overburden, and relatively high shear stresses were developed at the core-shell interfaces.

Squier (1967, 1970) discussed in considerable detail the instrumentation results from El Infiernillo Dam and Netzahualcoyotl Dam in Mexico. Various modes of load transfer were mentioned. During construction of both these dams the cores tended to settle with respect to the upstream and downstream shells, giving rise to certain patterns of movements and stresses. When the reservoirs filled and wetted the upstream shells, these settled with respect to the cores causing a reversal of the shear stress mobilization at the upstream interfaces and correspondingly complex movements in the structures. Stress concentration due to non-uniform abutment slope was also apparent.

In what has come to be the classic example of the consequences of stress transfer in dams, Kjaernsli and Torblaa (1968) reported the development of horizontal cracks through the core of Hyttejuvet Dam causing leakage and requiring a remedial grouting program. Stresses in the core were so low that the hydrostatic pressure of reservoir water caused hydraulic fracturing. Although it may be quite difficult to visualize the mechanism by which the fractures occurred the fact is that they did occur, and the authors presented a possible explanation (Fig. 1.4).

Stress transfer and arching is a ubiquitous feature of soil behaviour. The finite element technique is potentially capable of shedding light on the mechanisms involved and the way stress transfer develops over the history of a project.

1.5 Status of Finite Element Analysis in Geotechnical Engineering

Much time and energy has already been expanded by numerous authors in assembling state-of-the-art reviews of this subject. The finest examples available to this writer's knowledge are the series of addresses to the W.E.S. Symposium on Applications of the Finite Element Method in Geotechnical Engineering held at Vicksburg, Mississippi, in May 1972. It is easily obvious from the literature now available on the subject that quite enough techniques are available, but it is still necessary to prove the usefulness of the finite element as a design tool. Practical situations can often be solved using the simplest methods which will yeild reliable results of appropriate accuracy, if the analysis is based upon a realistic description of soil behaviour.

Few authors have reported successful usage of the method during design of a project. In only a very few of these were the analyses carried out prior to or early in the construction period. So one good question which practising engineers rightfully ask is "just how were the analyses performed?" It is always much easier to calculate the required answers after an event has occurred. The need for finite element research aimed at developing useful design tools is to predict behaviour before it happens. It is to be expected that the method will not be so successful as it appears now. How will consolidation effects be represented? How can representative deformation parameters be selected taking into account the effects of compaction machinery? Will simple (elastic?) parameters, carefully chosen using reasonable assumptions, give good enough answers or will sophisticated models using great arrays of test data be required? These are only some of the questions, and it is emphasized that they are only poorly answered after the event. Prior to application in design work some experience has to be accumulated in the analysis of case histories. The principle of performing "post mortems" is a natural development in any field where new concepts are being implemented in practice.

Far from being content that we now have a powerful new design tool, we should devote as much effort as possible to documented case histories where the analysis has been done prior to construction. Skermer (1974) indicated that 1969 analyses performed for Mica Dam produced construction movements larger than were measured at the site. The most important areas of development of the finite element method in geotechnical engineering must be in improving our predictive capabilities. ¶

1.6 The Finite Element As a Predictive Tool in Design

The finite element method can be used to compute pore pressure due to construction, consolidation, and seepage under complex boundary conditions. It can also be used to determine stresses and displacements at various stages of construction, under complex boundary conditions, and with simple or esoteric stress-deformation behaviour of many materials.

At the design stage where configurations and economics are being evaluated many analyses may be required. Computer budgets may be relatively restrained so only very simplified analyses could be undertaken. There is considerable argument for one branch of research effort to develop influence factors for the combination of materials and configurations usually considered in design, as suggested by Covarrubias (1969). When a design is being finalized special analyses will be required, but many simplifying assumptions would still need to be made particularly regarding stress-strain behaviour of the materials.

A

We are not yet at the stage where most simplifications can easily be justified. There is a great need for further research to explain just what effect certain simplifications will have. There is, above all, a great need for the persons who use the method in design to present their analyses and results, good and bad, in order to further confidence in the capabilities and limitations of the method in future design work. Otherwise there is the danger of finite element analyses being taken too seriously without good reason, or not seriously enough, again without good reason.

Significant contributions to the development of the finite element method to investigate cracking of earth dams have been made by Eisenstein, Krishnayya, and Morgenstern (1972, a,b) and Krishnayya (1973a). The 3D finite element program developed by Krishnayya (1973b) has been adapted by the writer to the analysis of construction of Mica Dam. The size and height of this structure, its location in a skewed steep river valley, and the materials used in construction all make it an excellent case to study and it is hoped the results of these analyses will contribute to the understanding of such a situation in future engineering works.

CHAPTER II

INTRODUCTION TO THE MICA DAM PROJECT

2.1 General Description of Mica Dam

Mica Dam is a high earthfill structure sited in a narrow gorge where the Columbia River leaves the Rocky Mountain Trench about 80 miles north of Revelstoke, British Columbia. The reservoir created will extend for over 130 miles in the Trench, from Golden to Valemount. 12 million acre-feet of live storage will be provided under the terms of the Columbia River Treaty (1964), and eventually 2500 MW of electrical power will be generated for Canadian consumption. A general description of the project has been given by Meidal and Webster (1973).

The damsite is in rugged, glaciated, mountainous terrain. Bedrock is a Precambrian metamorphic series of mica shist, granitic gneiss, pegmatite and white marble (in decreasing order of occurrence). Isoclinal folds are present, with bedding and foliation planes generally dipping downstream (southwards) at about 20°. Pleistocene geology is represented by glacial deposits and volcanic ash layers, river terraces and slope deposits. The sound bedrock conditions would be suitable for various types of dam, but after costs and material performances were evaluated a zoned gravel and rockfill structure with a near-vertical central impervious glacial till core was chosen. The arrangement of the fill is shown in Fig. 2.1. The crest zone was widened considerably after design conditions were revised during construction.

The dam axis is curved upstream on a 10,000 foot radius arc, and located so as to optimise wedging action between the abutments. Under reservoir loading, this should give extra safety against tension cracking of the core.

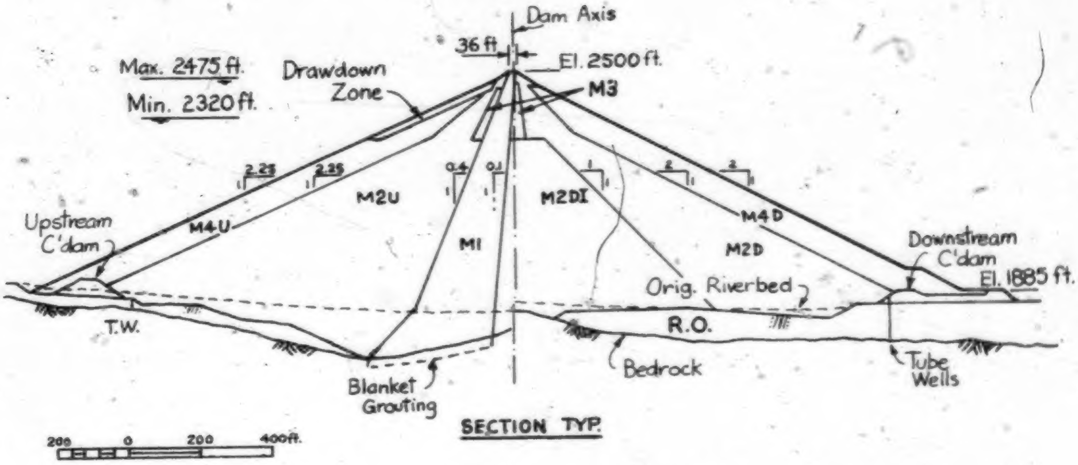
Surveying of the structure was based on a local coordinate system in terms of stations on and off-sets from the long chord.

Adequate safety against normal and drawdown conditions, and also severe earthquake loads, was provided ~~in~~ the design. Basic features, some later revised, are shown in Table 2.1.

2.2 History of Dam Construction

Twin 45 foot diameter concrete-lined tunnels were constructed in the left abutment between 1965 and 1967. The river was diverted in November 1967 after construction of the closure dykes, and cofferdam construction, riverbed excavation, core contact zone grouting and minor fill placement continued until March 1969, when the major fill placement program commenced. Dewatering of the construction area was by tube wells which were phased out when the fill height was beyond any danger of overtopping.

Some foundation layers of fine sand in the riverbed exhibited non-dilatant behaviour under high stresses during testing. Thus, all fine sand was removed from highly stressed


Dam Axis
36 ft
El. 2500 ft.
M3
Max. 2475 ft.
Min. 2320 ft.
Drawdown Zone
2.25
2.25
0.4
0.1
1
1
1
1
2
2
1
1
M2U
M2DI
M4U
M4D
M1
M2D
Upstream C'dam
Downstream C'dam
El. 1885 ft.
Orig. Riverbed
R.O.
T.W.
Bedrock
Tube Wells
Blanket Grouting
SECTION TYP.
200
0
200
400 ft.

ZONE	DESCRIPTION
M1	Core, glacial till in 25 cm (10") layers.
M2	Main shell, sand and gravel in 30 cm (12") layers, changed during construction to 45 cm (18") layers.
M2DI	Inner zone of poorer M2 materials.
M3	Core support zone, sand and gravel or rock in 15 cm (6") layers.
M4	Outer shell, sand and gravel or rock in 60 cm (24") layers.
Drawdown Zone	Gravel, cobbles and boulders or rock in 60 cm (24") layers.
R.O.	Original River Overburden

FIGURE 2.1 FILL ARRANGEMENT AS DESIGNED

TABLE 2.1

BASIC DAM DATA (after CASECO report)

	Metric Units	FPS Units
Elevations: Crest	762 m	2,500 ft
Riverbed	563 m	1,850 ft
Lowest bedrock	518 m	1,700 ft
Tailwater	573 m	1,880 ft
Drawdown range	754 to 707 m	2,475 to 2,320 ft
Height above lowest bedrock surface	243 m	800 ft
Height above riverbed	198 m	650 ft
Normal minimum freeboard	8 m	25 ft
Crest length	792 m	2,600 ft
Radius of crest curvature	3,048 m	10,000 ft
Crest width	11 m	36 ft
Heel to toe length	946 m	3,100 ft
Total volume of fill	32×10^6 m^3	42×10^6 yd^3
Total volume of core	3.3×10^6 m^3	4.3×10^6 yd^3
Slope of upstream face	1 on 2.25	1 on 2.25
Slope of downstream face	1 on 2.00	1 on 2.00

areas of the foundation because of the possibility of liquefaction under earthquake loads. Coarser overburden and till was left in place except in the core contact area.

Major fill placement occupied four construction seasons roughly from April to November each year, depending on climatic conditions, and was essentially completed by November 1972. The average placement properties of the materials shown in Fig. 2.1 are listed in Table 2.2, and it will be noted that significant changes were made after the 1969 construction season. Design properties quoted for the dam materials are given, for comparison, in Table 2.3. The different fill properties have important consequences detailed later on in this thesis. A comment should be made concerning zone M2DI. In this zone the contractor was allowed to place fill which was available much closer to the site. These materials were generally inferior to zone M2. As a result of the instrumentation program it can be stated that in general the materials were of high quality and good performance, and given the precautions taken in design (Webster, 1970), that the structure is sound and successful.

A concrete spillway structure, outlet works, and inlet facilities for future power development were constructed as fill was placed. An underground power station is at present under construction in the right abutment. An extensive survey network will monitor regional movements, and the University of British Columbia is managing a seismic

TABLE 2.2

FILL PLACEMENT STATISTICS 1969 - 1971 SEASONS

Year	Material	γ_d(pcf)	w(%)	γ(pcf)
1969	M1	131.1	8.8	143.0
	M2	146.1	7.6	158.0
	M2DI	144.4	7.6	156.0
1970	M1	141.5	6.3	150.4
	M2	146.6	3.7	152.0
	M2DI	145.6	4.7	152.4
1971	M1	141.5	6.2	151.0
	M2	146.5	3.6	152.0
	M2DI	145.6	4.7	152.0
1972	Not available 1972 materials assumed to be comparable with 1971			

TABLE 2.3
DAM DESIGN PROPERTIES
(after CASECO report)

			River Overburden	Shell Sand & Gravel	Core Glacial Till
Strength parameters:					
	ϕ'_{cu}		-	-	30°
	ϕ'		34°	38°	34°
	c'		0	0	0
	$\bar{B}$		-	0.2 end of construction	1.0 drawdown
Densities:	Moist	tonne/m^3	-	2.36	2.32
		lb/ft^3	-	147	145
	Saturated	tonne/m^3	2.24	2.42	2.37
		lb/ft^3	140	151	147
	Submerged	tonne/m^3	1.25	1.43	1.38
		lb/ft^3	78	89	86
Permeability		cm/sec	1×10^{-3}	1×10^{-3}	1×10^{-7}

array studyng the reservoir area.

2.3 Design Revisions During Construction

Placement Criteria. Following the first construction season important changes were made in the addition of fill. In the shell zones layer height was increased without significant change in compacted dry density, but the reduction in moisture content lowered the bulk density somewhat. Little change was caused to the material stiffness. In the core zones, the moisture content was reduced from near optimum (modified proctor) to 2% dry of optimum. Appreciable gain in compacted dry density was achieved and the core material became much stiffer than in the 1969 season.

The zone M2DI comprising "weaker" sand and gravel actually varied quite widely in properties, depending upon the source of the fill at any particular level. Further discussion is made of this variation in Section 4.5 where the derivation of material properties is discussed in detail. For convenience this zone was treated as if it, too, was distinctly different after the 1969 season.

Widening of the crest. When it was realized that a large slide or earthquake could cause a wave to overtop the dam careful consideration was made of the erosion hazard to the structure. Above elevation 2320 ft (707 m) the downstream slope was increased to 1 on 1.5, and the crest width adjusted to 111 ft. (33.8 m), and details of the revised crest geometry are shown on Fig. 2.2. With this arrangement erosion damage was not likely to cause general failure.

2.4 Instrumentation Program at the Dam

CASECO has produced detailed reports on the instrumentation program for each year since the program started. Webster and Lowe (1971) provide a concise summary of the instrumentation of Mica Dam and comment generally on instruments which are available for monitoring high embankments.

Several vertical or near vertical movement (MV) gauges provide the data on settlements, strains, and lateral movements which were extensively used in this investigation. Locations of these devices are shown on Fig. 2.3. They consist of telescoping sections of grooved casing installed with backfill in the embankment. Settlements were read using a field-developed latch-cone device or a radiosonde, while lateral movements were measured with a slope indicating instrument developed by SINCO of Seattle. Some details of installment of the MV gauges, and of the latch-cone device, are given in Appendix C.

Despite some failures for various reasons, the piezometers functioned well. Earth pressure cells yielded disappointing results in general (Skermer, 1974). Strain gauge

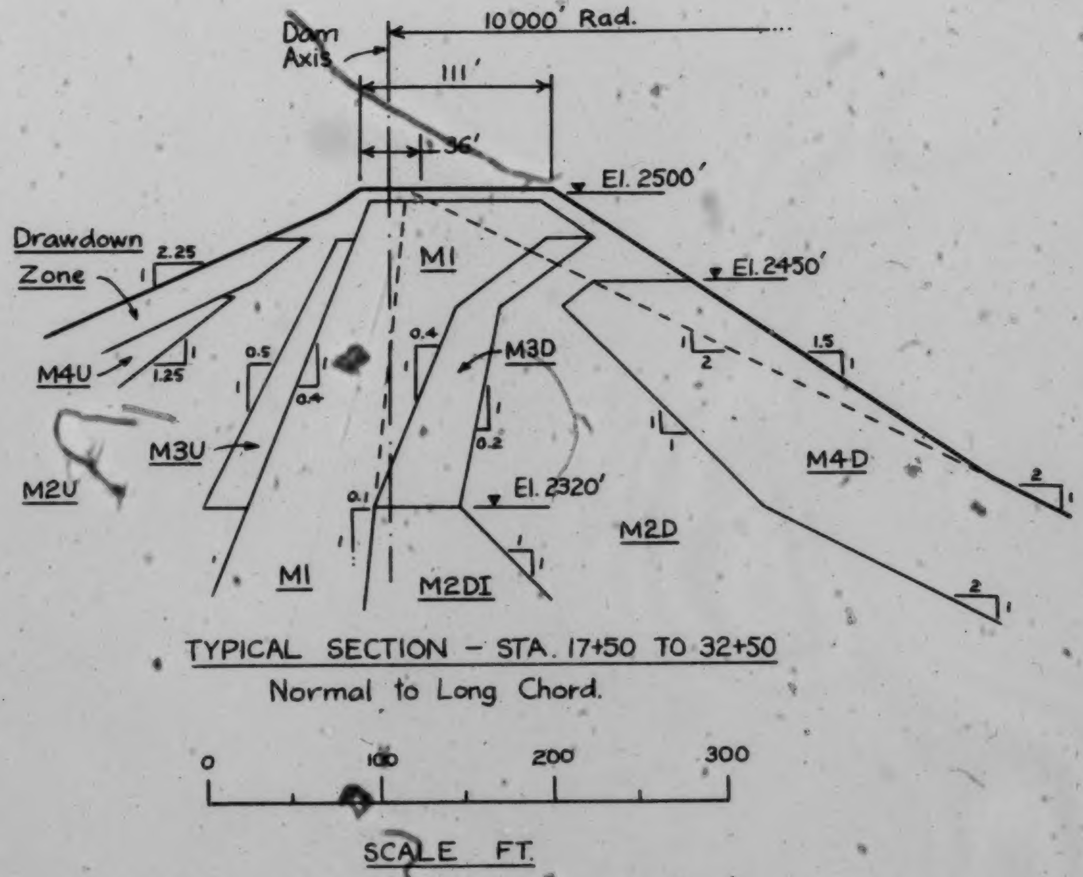

10 000' Rad.
Dam Axis
111'
36'
El. 2500'
El. 2450'
El. 2320'
Drawdown Zone
2.25
1
M4U
1.25
0.5
0.4
M3U
M2U
M1
M3D
0.2
0.1
M2DI
M2D
M4D
1.5
2
TYPICAL SECTION - STA. 17+50 TO 32+50
Normal to Long Chord.
0
100
200
300
SCALE FT.

FIGURE 2.2 CREST WIDENING DESIGN DETAILS

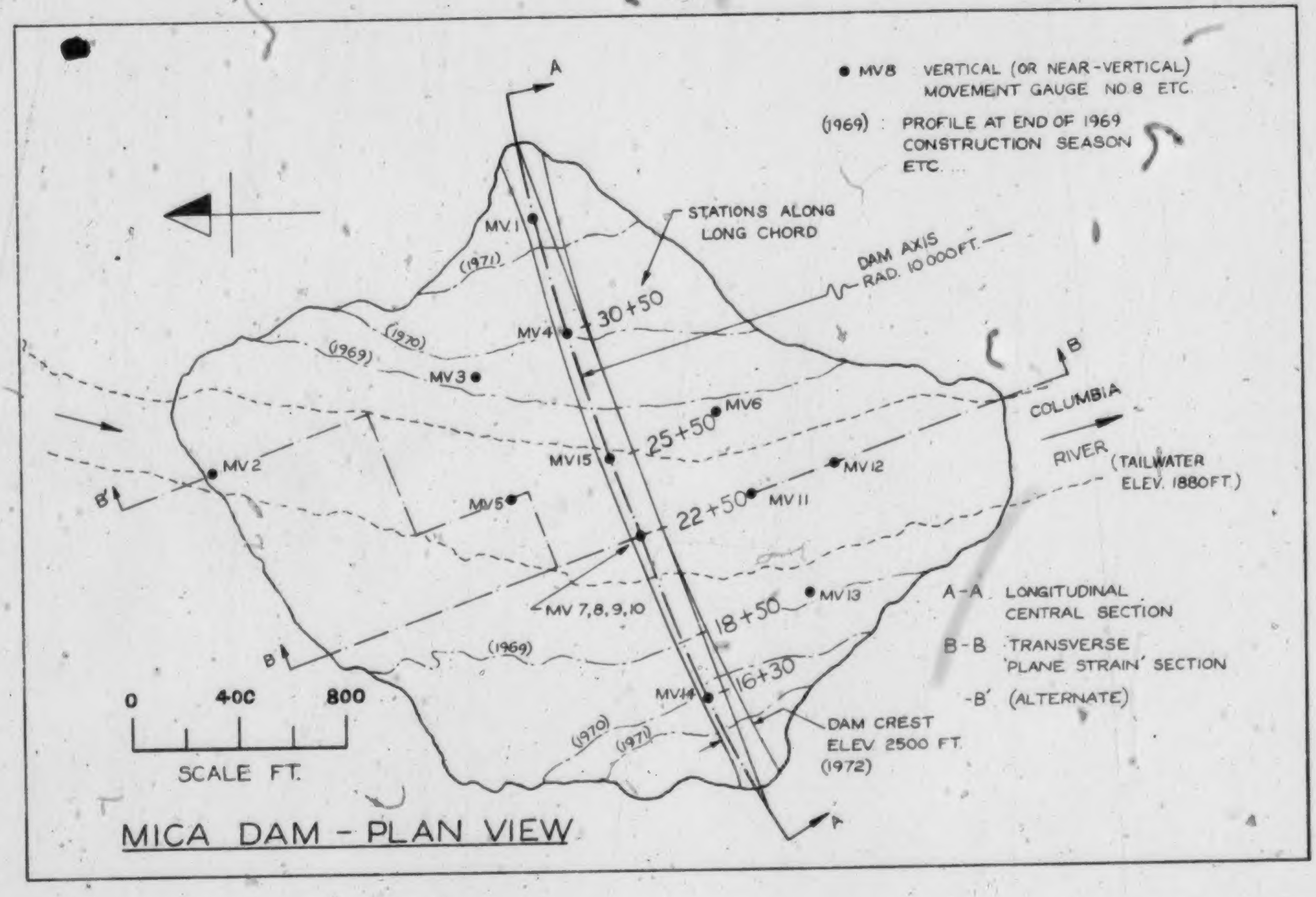

● MV8 : VERTICAL (OR NEAR-VERTICAL) MOVEMENT GAUGE NO.8 ETC.
(1969) : PROFILE AT END OF 1969 CONSTRUCTION SEASON ETC. ...
STATIONS ALONG LONG CHORD
DAM AXIS RAD. 10 000 FT.
A
A
B
B
B'
MV1
MV2
MV3
MV4
MV5
MV6
MV 7, 8, 9, 10
MV11
MV12
MV13
MV14
MV15
(1971)
(1970)
(1969)
(1969)
(1970)
(1971)
30+50
25+50
22+50
18+50
16+30
COLUMBIA
RIVER
(TAILWATER ELEV. 1880 FT.)
A-A LONGITUDINAL CENTRAL SECTION
B-B TRANSVERSE 'PLANE STRAIN' SECTION
-B' (ALTERNATE)
DAM CREST ELEV 2500 FT. (1972)
0
400
800
SCALE FT.
MICA DAM - PLAN VIEW

FIGURE 2.3 LOCATION OF MV GAUGES

results were ignored until after construction machinery had finished working, due to "noise" produced by dynamic loading from the moving construction equipment. Settlements and incremental strain data from the MV gauges can be used with confidence, but due to the very small horizontal movements in the embankment during construction the horizontal movement data from the MV gauges must be interpreted in a qualitative manner only due to the limits of instrument accuracy. Despite some shortcomings, the program provides far more data than can readily be assimilated, and is quite successful and useful for monitoring the structure or checking analyses.

CHAPTER III

FINITE ELEMENT ANALYSIS OF EMBANKMENTS

3.1 2D Versus 3D

Prior to the development of electronic computation techniques, rigorous stress-strain analysis of continua was confined nearly always to a 2D simplification because of the enormous complexity of the mathematics. Even then, solutions were developed only for certain cases of over-simplified geometries. With the advent of the direct stiffness method, the finite element concept, practical matrix algebra and digital computers, great steps forward in analytical power were made. Even so, simplification to a 2D state is still almost mandatory because of the logistic and economic problems of a 3D analysis.

Clough (1960) first used the term "finite element method " in relation to 2D plane stress analysis. Using the 2D state very complex analyses have been made and in soils engineering developments have in many ways completely outstripped any practical applications. Soils and rocks are complex materials, in general our limitations are not in analyses *per se*, but in understanding and being able to measure basic behavioural characteristics (if these are recognized) of the materials. Finite element 2D analyses are now capable of answering most of the questions that may be placed before them, and a matter of increasing concern is whether the 2D result is good enough in a 3D world. Experience suggests it is, in most cases, but only by evaluation of the two approaches can the question be truly answered.

2D programs can be run on modern computer facilities at a very nominal cost, considering their power and implications. With 3D programs, a very large sophisticated system is necessary to run even a modest analysis at a cost which may not be justifiable in terms of the additional information obtained.

The isoparametric finite element concept, powerful and generalized in 2D or 3D, has been widely adopted because of its versatility (Zienkiewicz, et al, 1969). Clough (1969) evaluated the most suitable element forms for 3D analysis. The most advanced system available at the time of writing, to the writer's knowledge, is documented by Bathe, Wilson, and Peterson (1973). In geotechnical engineering the most practical 3D results reported were provided by a program developed by Krishnayya (1973a).

Recent studies (Palmerton and Lefebvre, 1972; Eisenstein, Krishnayya and Morgenstern, 1972a) have clearly indicated conditions in which 2D analyses do not agree well with 3D analyses. The 3D situations are primarily a dam in a narrow, steep-sloped valley and dams with a thin central or inclined core of markedly different properties from the shell. Mica Dam satisfies both of these. The increasing development of high fill dams in narrow and irregular valleys accentuates the need to know the 3D effects in finite element analyses, in order to more adequately predict important features such as cracking or hydraulic fracture.

Previous work (Covarrubias, 1969; Strohm and Johnson, 1971, Eisenstein, Krishnayya, and Morgenstern, 1972 a,b) has provided good broad outlines as to factors influencing cracking. The control of cracking in earth dams is of paramount importance and is possibly the prime use for finite element analyses which can successfully predict movements and stresses. It remains for a predictive 3D analysis of a real structure to be reported, and this thesis work may assist such developments.

3.2 Representation of Soil Stress-Strain Behaviour

The classic formulation of stress components in terms of strain components for an ideal, isotropic, elastic solid can easily be written in terms of the familiar Hooke's Law, using Young's modulus and Poisson's ratio. Young's modulus is a measure of normal-stress/strain response, while Poisson's ratio is a measure of lateral deformations associated with a particular directional deformation. Neither of these traditional parameters has any clear significance when we are dealing with stress states in a continuum. The Bulk and

Shear moduli, which can be derived from Young's modulus and Poisson's ratio (or vice versa), do have a physically separable significance with regard to octahedral normal and shear stress states respectively (Domashcuk and Wade, 1969).

Natural materials exhibit non-linear, anisotropic, stress-or-strain-path dependent stress-strain response. Only by the crudest approximations can they be treated as ideal, isotropic elastic materials, and often the stronger materials can be realistically represented in this simplest way under normal working stress conditions. Finite element techniques relate stress and strain fields through constitutive equations, usually based on the simplest forms of elasticity. These latter result in easy fast calculations which can yield good results in general terms (e.g. Covarrubias, 1969) but which have limited success as flexible predictive tools. The other extreme is the 21 independent coefficients relating 6 stress and 6 strain components at an infinitesmal point, all of which may vary in a complex way with material and with stress state.

Volume change due to shear stress alone, or rotation of principal stress axes, are one common deviation from ideal elastic behaviour in soils. Another is the incorporation of plastic deformations into elastic response. Unhappily, the present state of soil testing is such that only very limited data for an analysis invoking complex behaviour are available. Smith and Kay (1971) describe a more satisfying approach to soil deformation behaviour, but one which is still beset with problems of inertia of the discipline and mystifying appearance to most of the potential users.

The most successful approaches yet made to soil non-linearity assume piecewise linear incremental elasticity. Footings have been analysed using a Young's modulus derived from Triaxial tests, and an assumed Poisson's ratio (Radhakrishnan and Reese, 1969). Stresses and movements in Oroville Dam have been calculated using iterative techniques based upon Kondner's (1963) proposals for hyperbolic stress-strain relations for soils (Kulhawy and Duncan, 1970). Data for non-linear variations in Young's modulus and Poisson's ratio are obtained from triaxial tests and approximated by hyperbolic functions. Krishnayya (1973a) analysed cracking in Duncan Dam using Triaxial test data stored in stress invariant form, but limited Poisson's ratio to a maximum of 0.49. Penman and Charles (1973) assumed values of Poisson's ratio and used the concept of constant equivalent compressibility from 1D behaviour to account for changes in modulus with stress level.

As pointed out so rightly by Smith and Kay, the results of such analyses are surprisingly good considering their deviations from the real stress-strain response of the soils. It may well be that simple methods give really good results for typical embankments, and until time effects or unusual stress paths or failure is considered, there is no need for further complexification, just a hard-headed sense of the realities and limitations involved.

In this investigation of Mica Dam the Bulk and Shear moduli formulation, as proposed by Clough and Woodward (1967), is used. The effect of modulus change with stress level is introduced by allowing the One-dimensional modulus to vary with octahedral normal stress, and a generally constant value of Poisson's ratio is assumed in calculating tables of K and G versus σ_{oct} for a material. This method is more fully described in Section 4.5.

3.3 Shear Failure Behaviour of Soil

Kulhawy et al. (1969) suggested that after shear failure of an element further stress-strain response could be modelled by keeping K at its pre-failure value and decreasing G towards zero. Krishnayya (1973a) computed shear failure using the same criteria as Kulhawy et al, and reduced the Shear modulus to

$$G = \frac{K}{50} \quad \text{after failure.}$$

The value of K was determined by unconfined compression test data, when he used the stress-invariant triaxial data approach. Otherwise, as suggested by Kulhawy et al, K keeps its pre-failure value, and this is the approach used during the present investigation. Stress redistribution and plastic deformation are only partially represented by this formulation

(Smith and Kay, 1971). Consequently the results of shear failure in the reported analyses of Mica Dam should be viewed with caution.

3.4 Pore Pressure Effects

During fill construction pore pressures are generated by total stress changes and dissipated through drainage. Particularly in thin cores of silty clays orglacial tills there may be appreciable dissipation over the construction period. Koppula (1970) utilized the governing equation

$$C^1\{ \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} \} = \frac{\partial u}{\partial t} - \gamma\bar{B} \frac{\partial h}{\partial t}$$

where C^1 = 2D coefficient of consolidation

u = pore water pressure

γ = bulk density of the fill

$\bar{B}$ = $\frac{\Delta u}{\Delta \sigma_1}$ (Bishop, 1954)

$\frac{\partial h}{\partial t}$ = time rate of increase of fill height.

which can be applied to vertical cores quite accurately. Krishnayya (1973c) developed a 2D finite element program which solves this equation, and which will be used in conjunction with total and effective stress analyses in another part of the Research Project involving Mica Dam (Law, 1974). Chan (1971), applied the equation in finite difference form to simulate pore pressures in Mica Dam, quite successfully except for the effects when foundation tube wells were deactivated, which he could not allow for.

The difficulty in stress analysis is in simulating the effects of drainage, as the deformation parameters can vary quite significantly. Partial drainage can be dealt with by doing analyses for the two limits of no drainage and full drainage (e.g. Chang and Duncan, 1970). The correct stress-strain relations can be determined from laboratory tests that simulate the proper stress paths and partial drainage consistent with field conditions. This can be impractical. In general the most successful attempts use consolidated-undrained test results as a reasonable medium. Fully drained conditions correspond to consolidated-drained tests and undrained conditions to unconsolidated-undrained tests. Poisson's ratio will approach 0.5, the condition of incompressibility, only for total stress analyses under undrained conditions. The importance of the best possible simulation of drainage conditions has been emphasized by Duncan (1972) and Lowe (1972).

In the present work, the fill materials were all considered as free draining except in the core. Thus there was no distinction between effective and total stresses in these materials. In the core, laboratory tests indicated that only in the 1969 material were significant pore pressures likely to be generated, and a reasonable judgement was made to obtain representative moduli (Refer to Section 4.5).

3.5 Incremental Analysis of Fill Construction

Clough and Woodward (1967) demonstrated that incremental finite element analyses are required to calculate displacements in an earth fill that would be measured in the field. For a homogeneous dam, they indicate that stress distributions from a simpler "gravity turn-on" analysis are reasonably close to those of the incremental analysis. Since actual construction involves layers far too thin to be individually applied in a finite element analysis, investigations have been made of the detail required to gain respectably good results with the finite element approximation.

Kulhawy et al (1969) discuss the layering problem in considerable detail, and suggest a means for determining the minimum layering required for a suitable accuracy. Other factors such as overall mesh size, computer resources, job requirements etc. can be expected to play a role in economic decision-making. Naylor and Jones (1973) used a remarkably coarse mesh in a separate analysis of Llyn Brianne Dam and construction was simulated using only 4 layers and a high-order element. They report results as successful as Penman and Charles' (1973) detailed analysis.

Other, more subtle effects arise with layering analysis. To what extent should shear between placed material and new fill be considered? Clough and Woodward used a "reduced modulus" to allow for reduced shear interaction between a new layer and existing fill. However, several techniques are used in construction to key a new layer into the existing surface, particularly with more plastic materials which can actually shear to residual under construction equipment loads (Sherard et al, 1963). Another approach is to assign the properties of a dense liquid to the new increment of fill in an analysis. This is neither necessary nor realistic.

Two fundamental approaches seem to have developed in incremental analysis. It is useful to examine them for their own sake, although it is doubtful whether their results would be significantly different.

Approach (A):

1. The first layer of elements is placed and assigned stresses so that σ_1 is the overburden pressure at the centroid of an element and σ_2 and σ_3 are calculated using the coefficient of lateral pressure (which may be calculated from Poisson's ratio).
2. The structure stiffness is calculated and nodal loads are applied to the upper surface corresponding to the weight of the next layer. If necessary these steps are repeated using the "average stress" approach.
3. The next layer of elements is added and stresses assigned in these as before. The stresses and

displacements due to the next layer of fill are then added to the previous results. The process is repeated until all fill loads have been added.

This approach is exemplified by Kulhawy and Duncan (1970) and Palmerton and Lefebvre (1972).

Approach B:

1. The first layer of elements is added and stresses and displacements due to self-weight of this layer are calculated using the layer stiffness.
2. The next layer of elements is added and stresses and displacements for this structure are accumulated with previous results. If necessary the "average stress" iterative approach is used.
3. The process is repeated until all fill has been placed. Thus nodes along the top of the fill have displacements due to the self-weight deformation of the last-placed layer. This situation does not occur in placing and monitoring of field measurement devices. Hence, the construction displacements of nodes on the upper surface of the nth layer are computed by subtracting from their total displacements the displacements due to self-weight of layer n.

This approach is exemplified by Krishnayya (1973b) and is the method used in analysing Mica Dam using the program FENA 3D. Intuitively, for an infinite number of layers, this process would be the more correct. In the normal situation it is unlikely that there will be significant difference between the approaches.

Theoretical Studies Using Program FENA 3D: Eisenstein, Krishnayya, and Morgenstern (1972a) compared results of 3D analyses and 2D plane strain analyses of the longitudinal section of a dam in a 1:1 valley. The results are summarized again here because of the light they shed on results of the 3D Mica Dam analysis. All the analyses used linear properties and varying numbers of lifts.

1. There should be about 10 lifts to compute strains, stresses and displacements to agreeable accuracy using a 2D analysis. For a 3D analysis economic considerations may reduce the number of lifts, but this reduction should be as small as possible. Refer to Fig. 3.1.

2. Only for homogeneous embankments will a plane strain analysis of a longitudinal section in a steep valley yield satisfactory information about stresses and displacements. The effects of different stiffness in core and shell force us to use a 3D analysis to obtain satisfactory information (Fig. 3.2). Lefebvre et al. (1973) discuss in more detail the effects of 2D and 3D variations in homogeneous embankments.

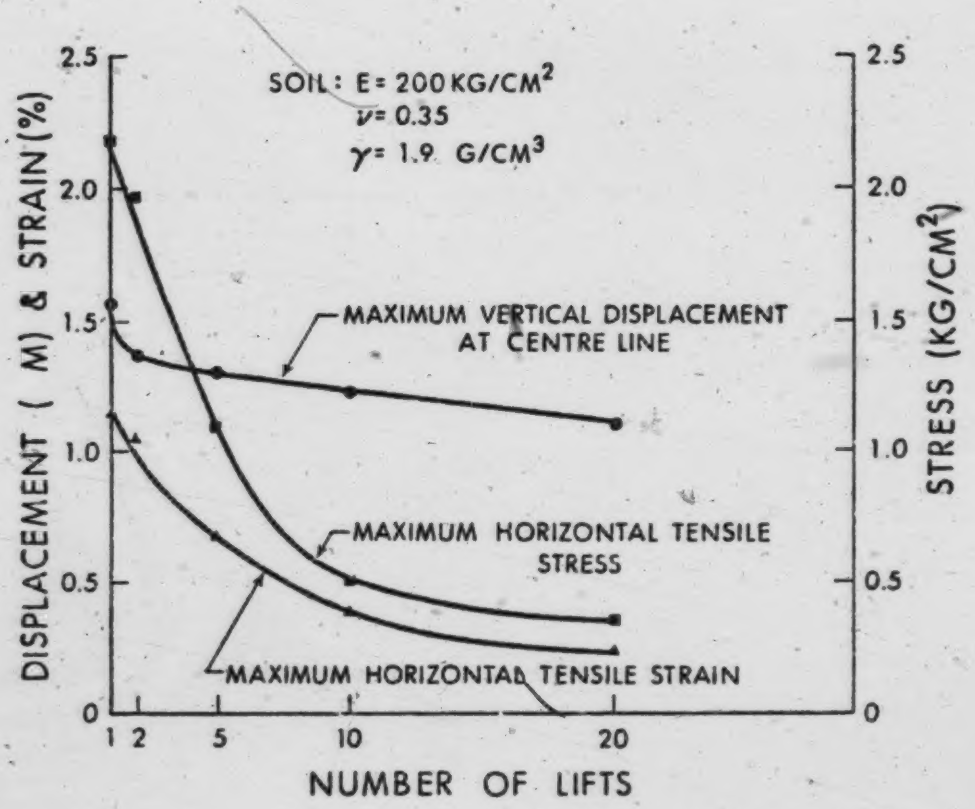


FIGURE 3.1 EFFECT OF NUMBER OF LIFTS ON SOME IMPORTANT PARAMETERS (after Krishnayya, 1973a)

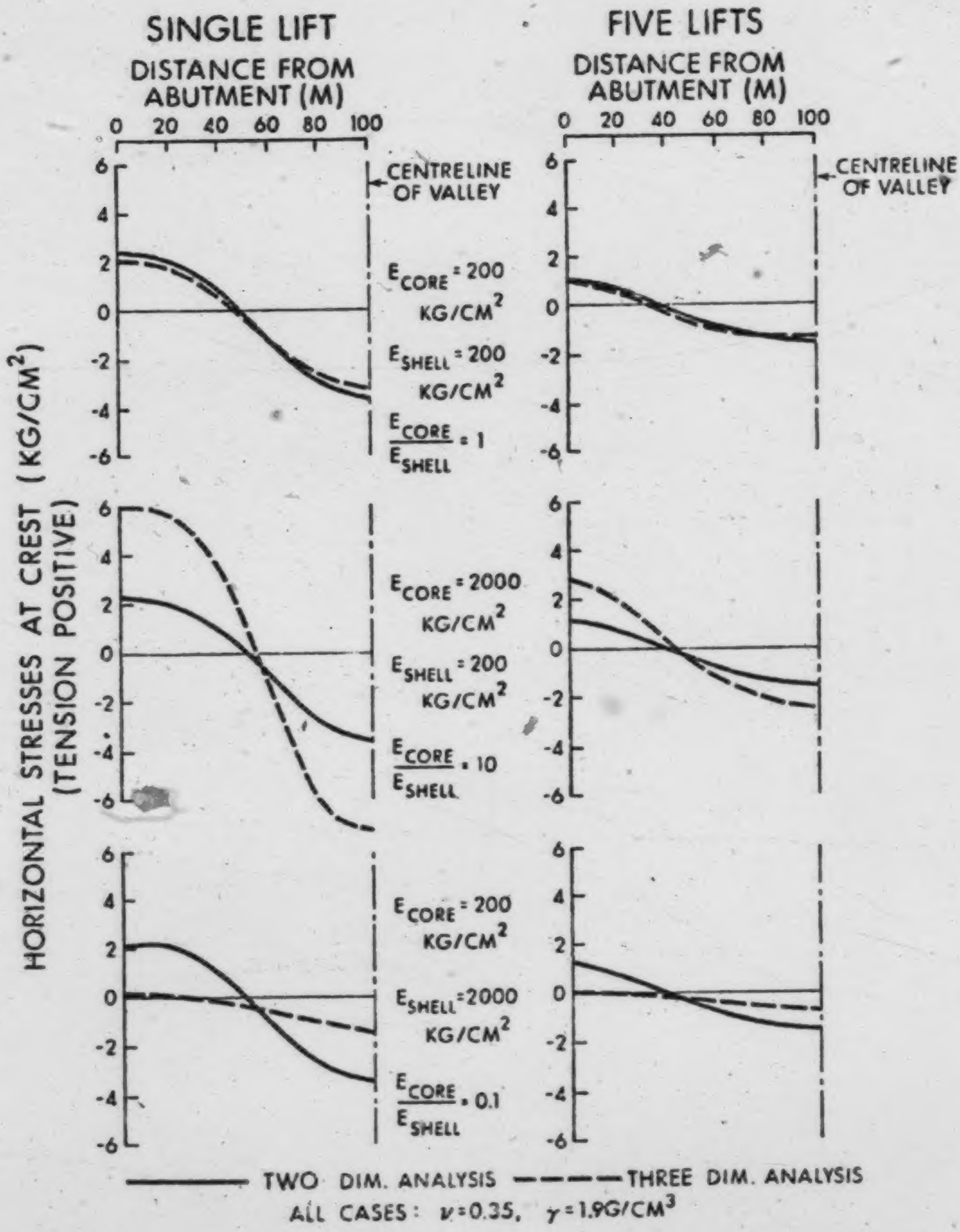


FIGURE 3.2 COMPARISON OF HORIZONTAL STRESSES ALONG A DAM CREST FOR 2D AND 3D ANALYSES (after Krishnayya, 1973a)

CHAPTER IV

FINITE ELEMENT MODELLING OF MICA DAM

4.1 Discussions and Site Visit

Discussions at various stages of the work described in this thesis always involved some aspect of the dam's character. This was felt to be invaluable to the writer as, over a period of time, this constant reference to the structure resulted in a confidence and familiarity with data which it is felt is very important when the time comes to discuss results. Comment on this aspect is included here because the writer regards it as a very important phase of finite element analysis in general. Particular emphasis must be placed on discussions with CASECO staff. A visit to the damsite was made in September, 1973 where features of the project were seen and discussed at first hand.

4.2 Construction and Instrumentation Drawings and Report

CASECO Consultants provided the following information on the dam structure and materials:

(a) Field Instrumentation reports for the 1969, 1970, and 1971 construction seasons, and a set of preliminary drawings from the 1972 report.

(b) Reports of triaxial tests performed by other agencies; of the core, shell, and river overburden materials.

(c) A selection of construction drawings showing the planned and actual excavations which occurred in the core zone.

(d) A series of fill progress drawings showing the transverse profiles across the valley.

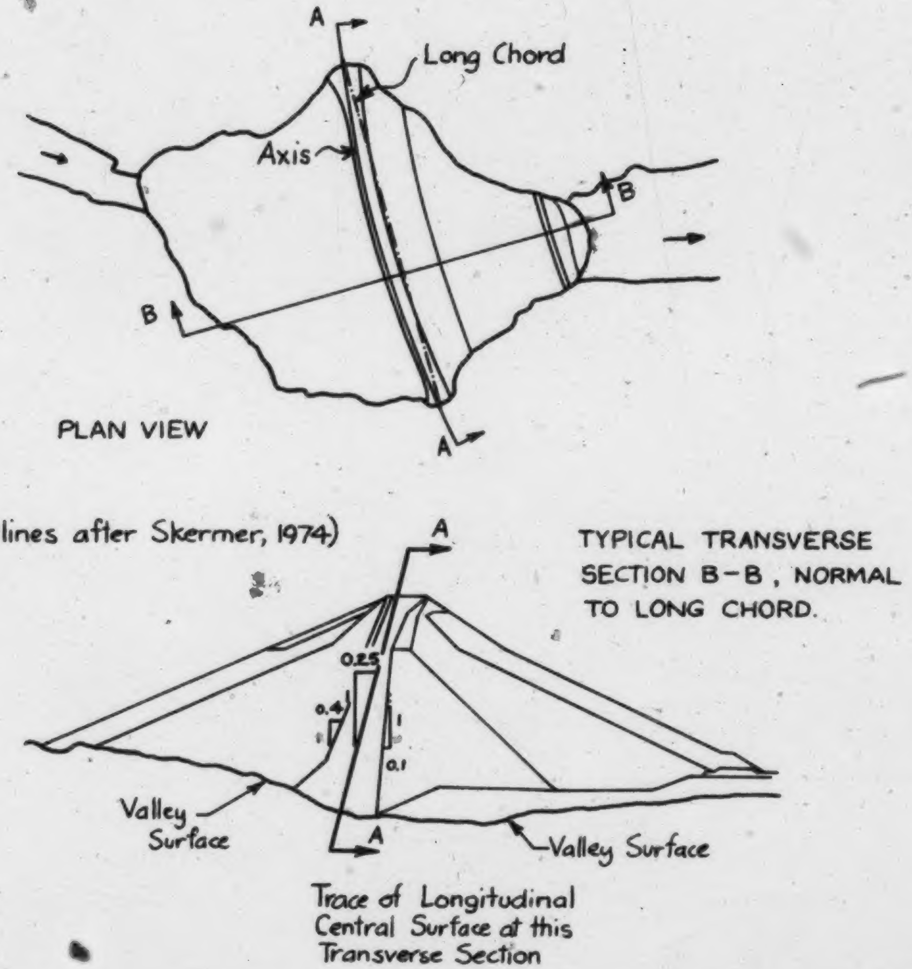
(e) Sundry details including some finite element analysis results, and construction drawings for the upstream and downstream cofferdams.

These resources were continually employed in determination of structure geometry and material properties.

4.3 Synthesis of Mica Dam Geometry

A master plan was prepared at a scale of 1 inch = 100 feet on which was reproduced the detail of all available construction information, the result being a contoured representation of the site after all overburden removal and core zone excavation was complete, just prior to the placement of fill.

Since the prime concern in developing a model of dam geometry was to successfully represent the slopes and slope changes of the abutments, a profile was prepared (from the master plan) of the intersection of the longitudinal central surface and the valley surface. (These terms are explained in Fig. 4.1) From this profile the important changes of slope were identified, and transverse abutment-surface profiles constructed at these points. Fill profiles at each of these locations were then added, so that the dam geometry


A
Long Chord
Axis
B
B
A
A
0.25
0.4
1
1
0.1
Valley Surface
A
Valley Surface

PLAN VIEW

(Outlines after Skermer, 1974)

TYPICAL TRANSVERSE SECTION B-B, NORMAL TO LONG CHORD.

Trace of Longitudinal Central Surface at this Transverse Section

The surface swept out by the sectioning lines A-A, in moving from station to station along the long chord, is termed the LONGITUDINAL CENTRAL SURFACE.

FIGURE 4.1 EXPLANATION OF VALLEY SURFACE AND LONGITUDINAL CENTRAL SURFACE

was described by a series of vertical sections transverse to the long chord.

The element mesh could then be assembled by subdivision within the vertical sections, having regard to abutment surfaces and internal structures of the fill. At this stage the two-dimensional visualization process broke down, and a model was constructed. The series of vertical sections were reproduced with coloured marking pencils on sheets of lucite, which were mounted in their correct positions on a base-board covered with a copy of the master plan. This arrangement, giving visual access to the 2D and 3D geometry, was invaluable at this stage and even more valuable at other stages, and the writer emphasizes that such a 3D model is absolutely essential for an analysis of this sort. Even so, a great amount of effort is necessary to mentally visualize small details. A view of the model is shown in Fig. 4.2.

4.4 Subdivision Into Finite Elements

The inaccuracies of poorly shaped and skewed elements become crucial at this point, as too large a subdivision will be too crude, too small a subdivision will be impossibly expensive, and anything between these limits must have satisfactory elements and representative geometry. The process requires considerable experience and since the time involved is measured in weeks or months, the writer emphasises that herein lies a cost at least as significant as computer costs.

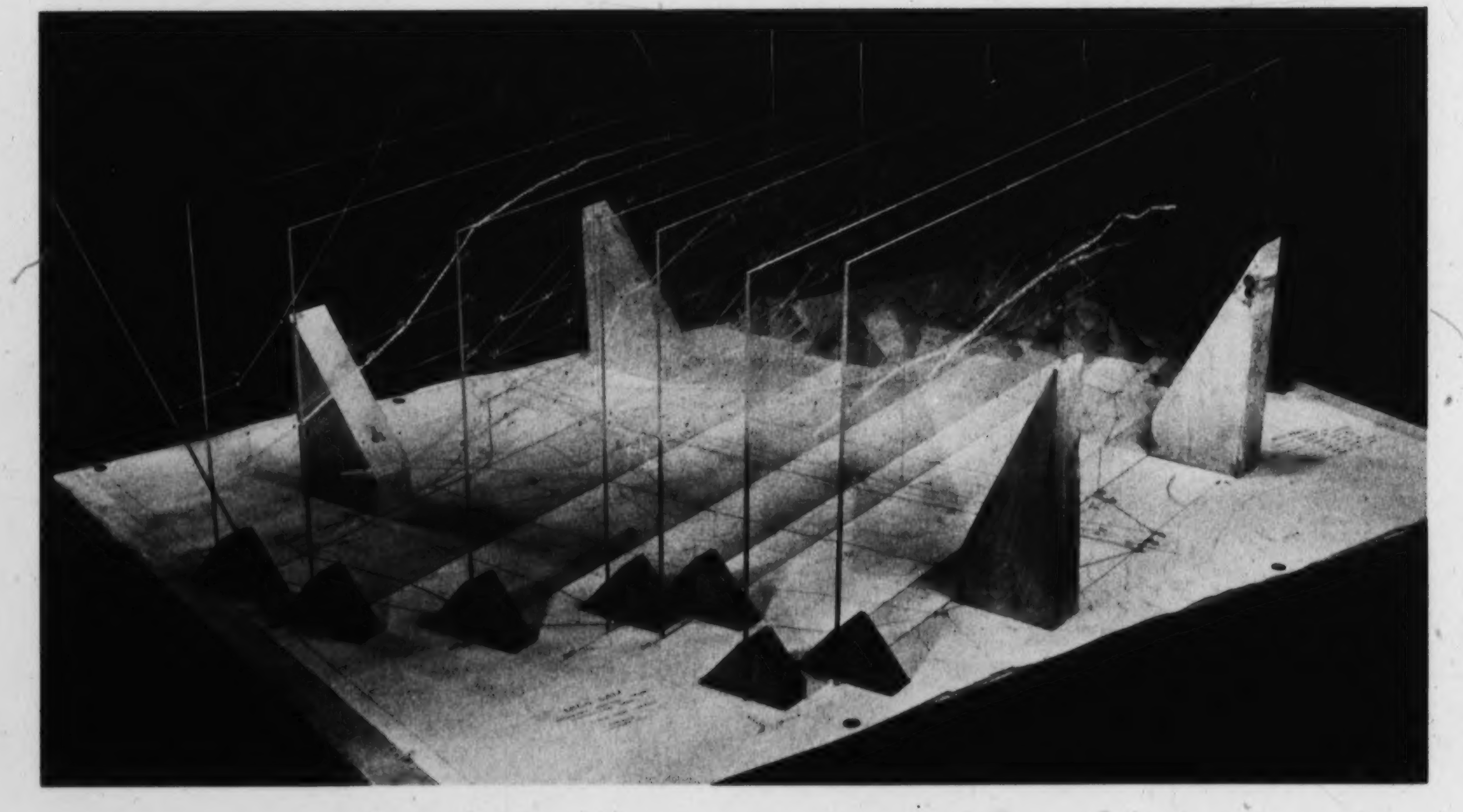


FIGURE 4.2 MODEL OF 3D MICA DAM FINITE ELEMENT MESH

A mesh regarded as "satisfactory" was finally developed. After considering the shape of elements there resulted a finite element mesh of 254 elements and 276 nodes, yielding a structural stiffness matrix of 828 equations with a semi-bandwidth of 183. After removing the boundary nodes (where some displacement condition was specified), there were 438 effective degrees of freedom. This mesh consisted of five lifts:

1. river overburden after excavation

2. to 5. fill placed in the 1969 to 1972 construction seasons respectively.

Since one element height thus represents one full season of fill placement this is the coarsest mesh possible. Even thus, this mesh is as large as could be conveniently used for repeated calculations, and so is the finest mesh possible.

The complexities involved meant that simulation of river overburden excavation was not attempted. Since these materials are probably highly overconsolidated this is a deficiency of the analysis. The results obtained for river overburden material were thus treated with extreme reserve, and in this area the worst element-geometry problems compounded the doubts. Further discussion on this topic can be found in Chapter V. The problem of the river overburden elements may have had significant effects on all calculations made, but it was felt that even so the overall results of analyses using this mesh would be quite realistic and representative.

During the process of subdivision several nodes were located between sections, using model and master plan. Extra elements were included to assist in modelling the river overburden materials. The mesh geometry is presented in Appendix B. Figure 4.3 shows the section at station 22+50, the main instrumentation plane and the assumed plane strain section.

4.5 Derivation of Material Properties

The program FENA. 3D contains an option to use stress-strain data from triaxial tests to interpolate elastic parameters for an element based on its stress state. This is an elegant but complicated and expensive way of performing the analyses. Accordingly most analyses were carried out using "engineering judgement" to select the "representative" linear elastic moduli. As pointed out by Skermer (1974) it is very difficult to obtain laboratory data which match field behaviour for soils, because of compaction and grading problems chiefly. In most cases incremental stress-strain data from the MV gauges were used. Laboratory tests by T. C. Law which are part of further research being undertaken at the University of Alberta, were available for core material. Reports of triaxial testing for CASECO were also available for some materials. A summary of available stress-strain

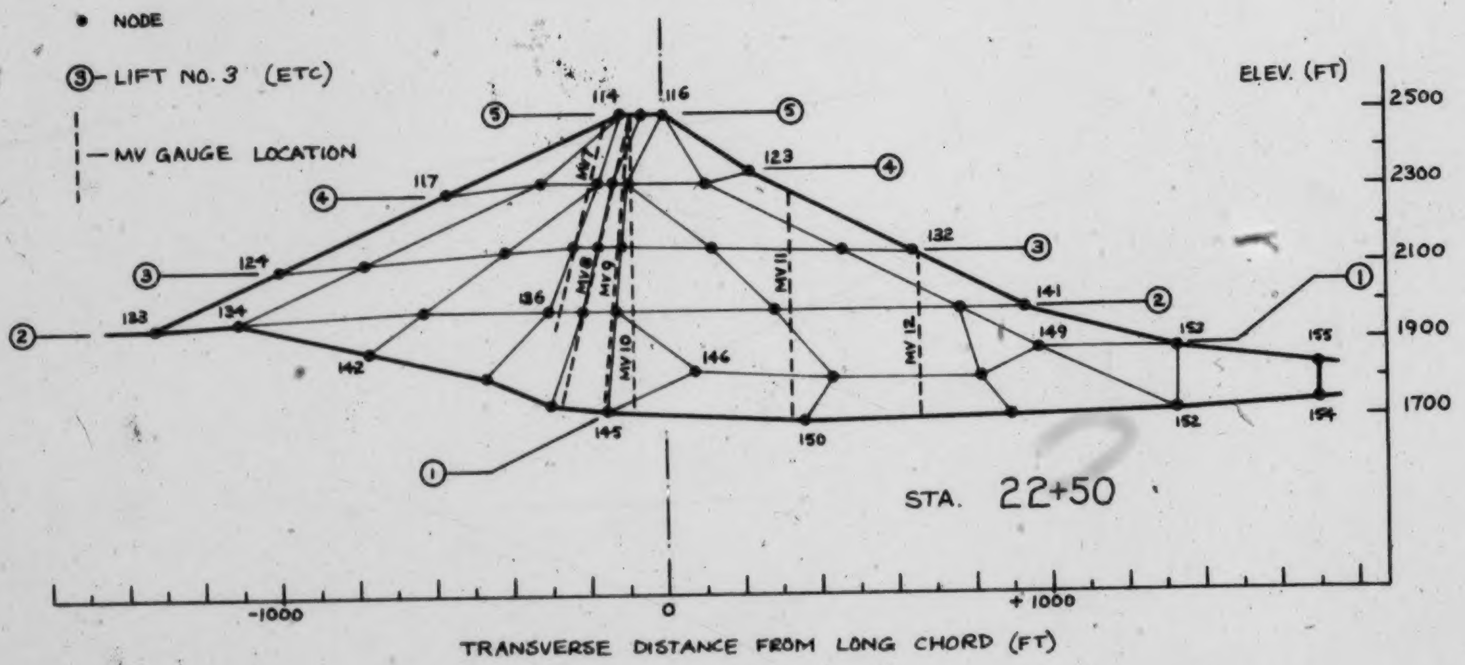

• NODE
③ - LIFT NO. 3 (ETC)
— MV GAUGE LOCATION
ELEV. (FT)
2500
2300
2100
1900
1700
114
116
117
123
124
132
133
134
136
141
142
145
146
149
150
152
153
154
155
MV 7
MV 8
MV 9
MV 10
MV 11
MV 12
①
②
③
④
⑤
STA. 22+50
-1000
0
+1000
TRANSVERSE DISTANCE FROM LONG CHORD (FT)

FIGURE 4.3 TRANSVERSE SECTION, STA.22+50:
FINITE ELEMENT MESH

information is given in Table 4.1.

An approach towards non-linear behaviour of the soils was made by allowing moduli to vary with stress level in the elements, as described hereunder. As previously mentioned, elastic, isotropic materials possess two independent stress-strain parameters. The usual form is Young's modulus (E) and Poisson's ratio (μ). An alternative formulation uses the Bulk modulus (K) and Shear modulus (G). In this work the One-dimensional modulus (D) is ubiquitous. In general, all these parameters vary in their own way with variations in the stress state. Assuming isotropy and linear elasticity the following interrelationships exist between the variables:

$$K = \frac{E}{3(1 - 2\mu)}$$

$$G = \frac{E}{2(1 + \mu)}$$

$$E = \frac{D\,(1 - \mu - 2\mu^2)}{(1 - \mu)}$$

Poissons' ratio: In general Poisson's Ratio will vary depending upon the stress state of the soil. Computation of horizontal movements and minor principal stresses is more sensitive to variations of this variable than settlements and major principal stress (Skermer, 1973; Kulhawy et al, 1969; Covarrubias, 1969).

TABLE 4.1

SOURCES OF STRESS-STRAIN INFORMATION

Material Description	No.	University of Alberta Tests	U. of A. Gauge Summary	CASECO Triaxial Reports	CASECO Gauge Summary
River Overburden	1	-	good	poor	fair
Shell M2	2	-	good	fair	good
Core M1 (70-72)	3	very good	fair	fair	-
Shell M2DI (70-71)	4	-	fair	-	good
Shell M2DI (69)	5	-	poor	-	good
Core M1 (69)	6	very good	poor	fair	-

Assessments are qualitative in terms of relative value for the purpose

Skermer (1973) indicates the change of Poisson's ratio with stress state during an analysis, where non-linear triaxial data was used, and the principle is generally accepted. Results from the Mica 3D analyses (linear properties) indicate that the computed stress state for a material does not necessarily agree with the relation developed between σ_1 and σ_{oct} in Section 1.3. What deviation was due to stress transfer or computer roundoff errors, and what deviation may be an indication that the material should have exhibited a change in value of Poisson's ratio could not be determined. The deviation shown in Fig. 4.4 is for shell zone M2 material. The writer argues that the assumption of a reasonable value for Poisson's ratio is a satisfactory first-order approximation to the true behaviour.

One-dimensional compression: Particularly at the early stages of construction any lateral movements of a fill are negligible compared to settlements. It is satisfactory to assume one-dimensional compression for which the incremental modulus

$$D = \frac{\Delta\sigma_{vert}}{\Delta\varepsilon_{vert}}$$

This one-dimensional modulus, as shown earlier in this section, is nothing more than one of a set of two required constants for simple linear elastic analysis. It perhaps best describes the stress-path undertaken by a soil element during fill construction, particularly of high fills of stiff

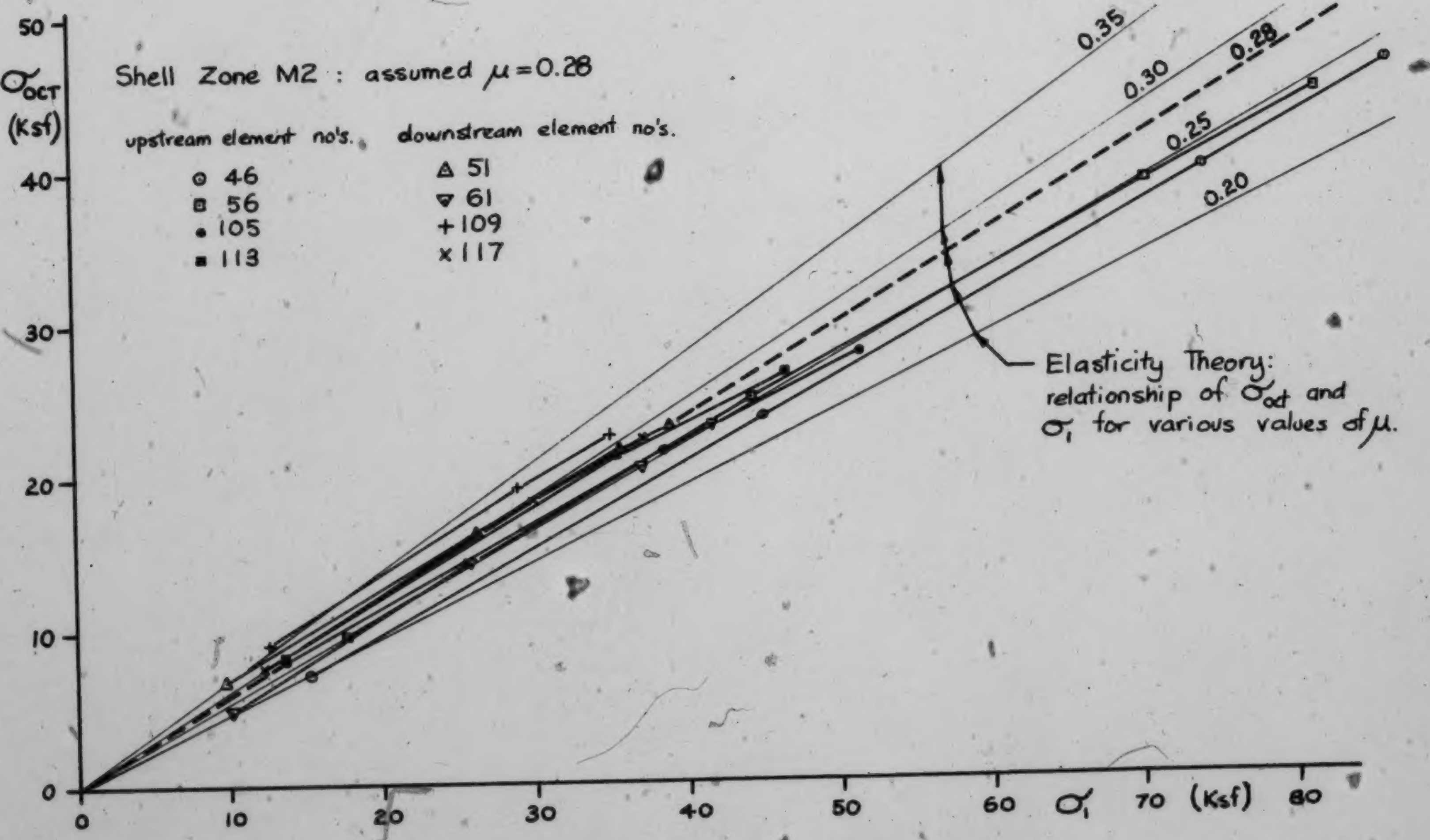


FIGURE 4.4 SHELL ZONE M2: COMPUTED AND THEORETICAL STRESS INTERRELATIONSHIPS

materials such as Mica Dam. In this structure lateral movements were very small and field measurements of the D modulus satisfactorily reflect the stress-strain behaviour during construction when used with an appropriate value of Poisson's Ratio. This modulus was invariably higher than equivalent moduli calculated from triaxial tests on the fill materials

Allocation of material properties: On the basis of field and laboratory data six different stress-strain modes were selected for the Dam materials. A seventh material was allocated due to the higher bulk density of the M2 shell material in 1969. The materials are:

1. River overburden
2. Shell M2 (sand and Gravel) 1970-1971
3. Core M1 (Glacial Till) 1970-1972
4. Inner Shell M2DI (Sand and Gravel) 1970-1972
5. Inner Shell M2DI (Sand and Gravel) 1969
6. Core M1 (Glacial Till) 1969
7. Shell M2 (Sand and Gravel) 1969.

Linear elastic parameters were selected as described hereunder. In all cases μ was chosen as a reasonable value, the most important influence being the relationships:

$$K_o = 1 - \sin \phi' \quad \text{(Brooker and Ireland, 1965)}$$

and

$$\mu = \frac{K_o}{1 + K_o} \quad \text{(linear elastic theory)}$$

Nonlinearity was introduced by allowing the moduli to vary depending upon stress level in the elements. It was assumed that the One-dimensional modulus varied with the octahedral normal stress in the element. From the field curve of vertical stress versus vertical strain a plot of the D modulus versus σ_{vert} was made. Using the relationships of σ_{vert} and σ_1, and σ_1 and σ_{oct}, obtained for the particular material by averaging the results of linear analyses of the dam , D was expressed in terms of σ_{oct}. Assuming appropriate (usually constant) values of μ a table of K and G moduli for various σ_{oct} levels was assembled for computing. As it turned out the theoretical relationship discussed earlier between σ_1 and σ_{oct} was sufficiently good to represent the finite element behaviour, so that a table of K and G versus σ_1 would have been just as appropriate. The process of obtaining the multilinear properties is shown in Fig. 4.5.

The multilinear approach thus enables the stiffness of an element to vary with stress state. It does not take into account dilatency of the materials nor the variation of Poisson's ratio in general. The stiffness is derived from one-dimensional stress-strain data and has similarities to the approach described by Penman, Burland and Charles (1971) and Penman and Charles (1973). In the program FENA.3D an "average stress" process (as described by Kulhawy et al, 1969, and used by Krishnayya, 1973b) was used to obtain the best stiffness at each layer of analysis.

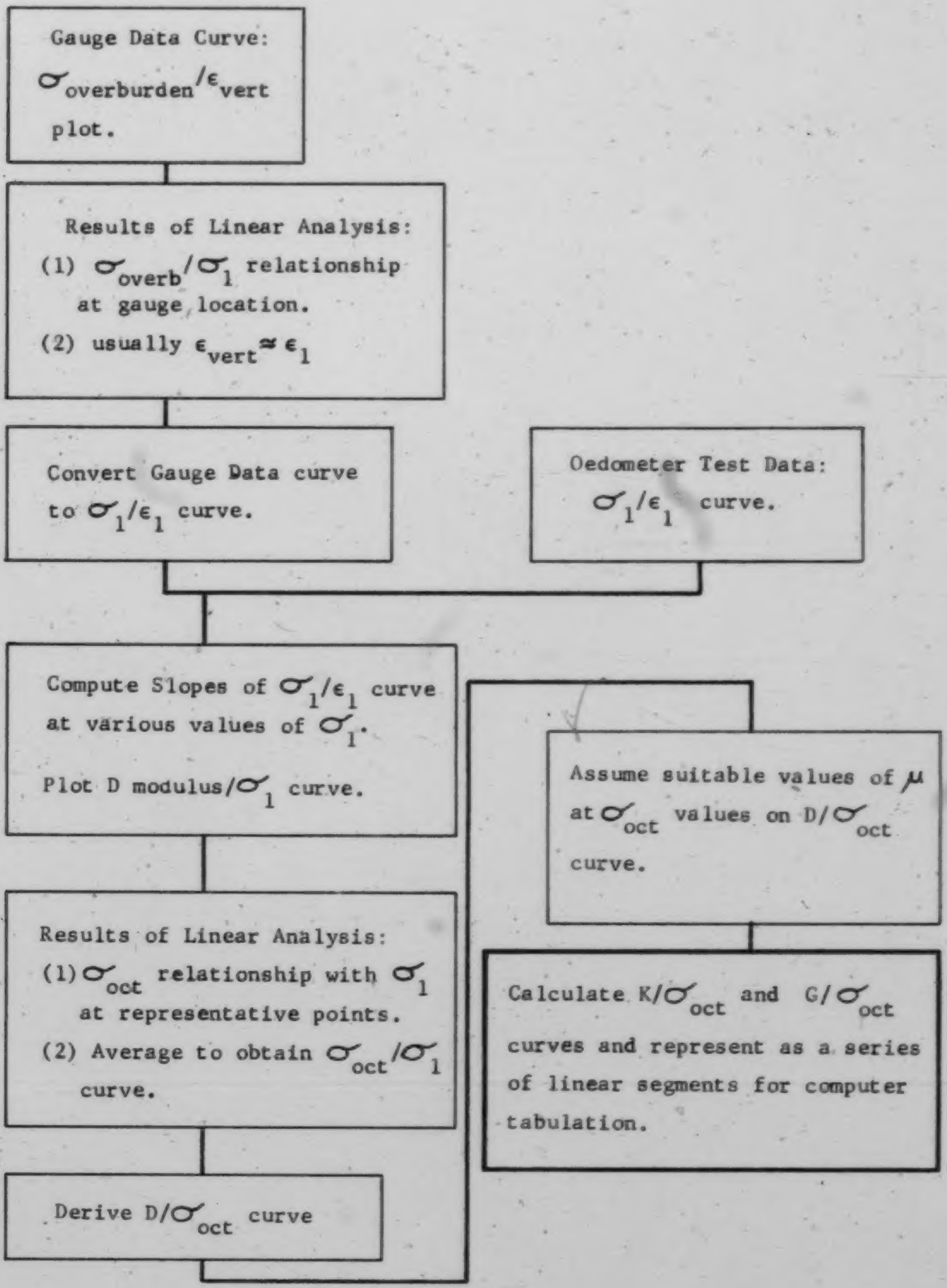

Gauge Data Curve: $\sigma_{overburden}/\epsilon_{vert}$ plot.
Results of Linear Analysis: (1) σ_{overb}/σ_1 relationship at gauge location. (2) usually $\epsilon_{vert} \approx \epsilon_1$
Convert Gauge Data curve to σ_1/ϵ_1 curve.
Oedometer Test Data: σ_1/ϵ_1 curve.
Compute Slopes of σ_1/ϵ_1 curve at various values of σ_1. Plot D modulus/σ_1 curve.
Results of Linear Analysis: (1) σ_{oct} relationship with σ_1 at representative points. (2) Average to obtain σ_{oct}/σ_1 curve.
Derive D/σ_{oct} curve
Assume suitable values of μ at σ_{oct} values on D/σ_{oct} curve.
Calculate K/σ_{oct} and G/σ_{oct} curves and represent as a series of linear segments for computer tabulation.

FIGURE 4.5 MULTILINEAR PROPERTIES SELECTION PROCESS

Properties used in the linear analyses are shown in Table 4.2 and the multilinear properties are shown in Table 4.3.

Core material: By coupling the results of high-pressure oedometer tests and all-round pressure tests it is possible to obtain two independent stress-strain relationships and hence to calculate two independent elastic parameters which can vary with stress state. Such work is currently being performed on the Mica Dam core material as part of further analyses within the Research Project, by T. C. Law (1974). While it was possible to incorporate this extra information into the 3D analyses, a decision was made to use simple judgement in assigning an essentially constant value of μ. This simplifies the selection of data for computation but remains to be justified by comparison of analytical and field data. The 3D analysis was felt to be too coarse to justify any more than simple approximations to the truly complex nonlinearity of soils. Nonlinearity is most evident in the finer grained core material, so in this region the greatest simplifications have been made. Some oedometer test data are shown in Figs. 4.6 and 4.7, and this one-dimensional data was used to select D moduli.

Shell zone M2: Field data was available from several MV gauges but MV11 and MV12 were deemed to give the best results, being influenced essentially by valley arching only. A plot of data used to calculate D moduli is shown in Fig. 4.8.

Shell inner zone M2DI: CASECO were able to supply a study of the variation of materials in this zone, which as described earlier was comprised of poorer materials from different sources. Their plot of the variation of moduli and materials is shown in Fig. 4.9. Skermer (1974) was able to model the distribution of these materials with greater detail in his finer 2D mesh.

River overburden: Triaxial tests were performed on this material. Field settlement data indicate a bulge at the overburden - shell interface, suggesting some disturbance during construction works. In general it is to be expected that these materials would behave as very dense, overconsolidated granular fill, and their insitu deformation properties would be correspondingly much stiffer than measured in a normal test. This factor was taken into account when selecting moduli, as the field data is necessarily quite limited for the river overburden. A plot of some gauge data is shown in Fig. 4.10.

4.6 Synthesis of Bedrock Movements

A tremendous volume of river overburden material was excavated for the core trench of the dam. Field measurements indicated that there was an unexpectedly large heave of the bedrock at this time, and that some heave actually continued as fill was placed during 1969. Rock settlements in this zone were greater than expected, probably due to loss of tightness in the rock mass due to heaving. Law (1974) correlated some bedrock heave with pore pressures in the rock, but not enough data is available from the excavation period to make conclusive arguments about the rock movements.

Krishnayya (1973a,b) analysed cracking at Duncan Dam, which was due to excessive differential foundation settlements, by adding observed foundation movements at appropriate boundary nodes of the finite element mesh. It was very easy to adapt the data on bedrock settlements at Mica Dam. By plotting the settlements at pertinent sections of the dam the incremental settlements for each lift of the analysis were determined and applied at the boundary nodes. Some interpolation and "guestimation" was required but in general the bedrock settlements are relatively very small and a representative pattern is a fairly simple process. For the larger settlements there was some concern as to accompanying horizontal rock movements and after some very simple analyses of rock slopes under fill load a rough guide was obtained, depending upon rock slope and position under fill. The horizontal movements were most likely too small to be influential and in retrospect it would have been simpler to ignore them completely. Longitudinal and transverse sections showing bedrock movements as measured at MV gauges are shown in Figs. 4.11 and 4.12 respectively.

TABLE 4.2

MATERIAL PROPERTIES FOR LINEAR ANALYSIS

Material	No.	Unit Weight (Kcf)	D(ksf)	μ	E(Ksf)	K(Ksf)	G(Ksf)
River Overburden	1	0,152	11000	0,31	7940	6960	3033
Shell Zone M2 (70-72)	2	0,152	9800	0,28	7670	5810	3000
Core Zone M1 (70-72)	3	0,150	4100	0,33	2767	2710	1040
Shell Zone M2DI (70-71)	4	0,152	7350	0,29	5640	4480	2186
Shell Zone M2DI (69)	5	0,156	8400	0,29	6450	5120	2500
Core Zone M1 (69)	6	0,143	2000	0,35	1250	1390	464
Shell Zone M2 (69)	7	0,158	9800	0,28	7670	5810	3000

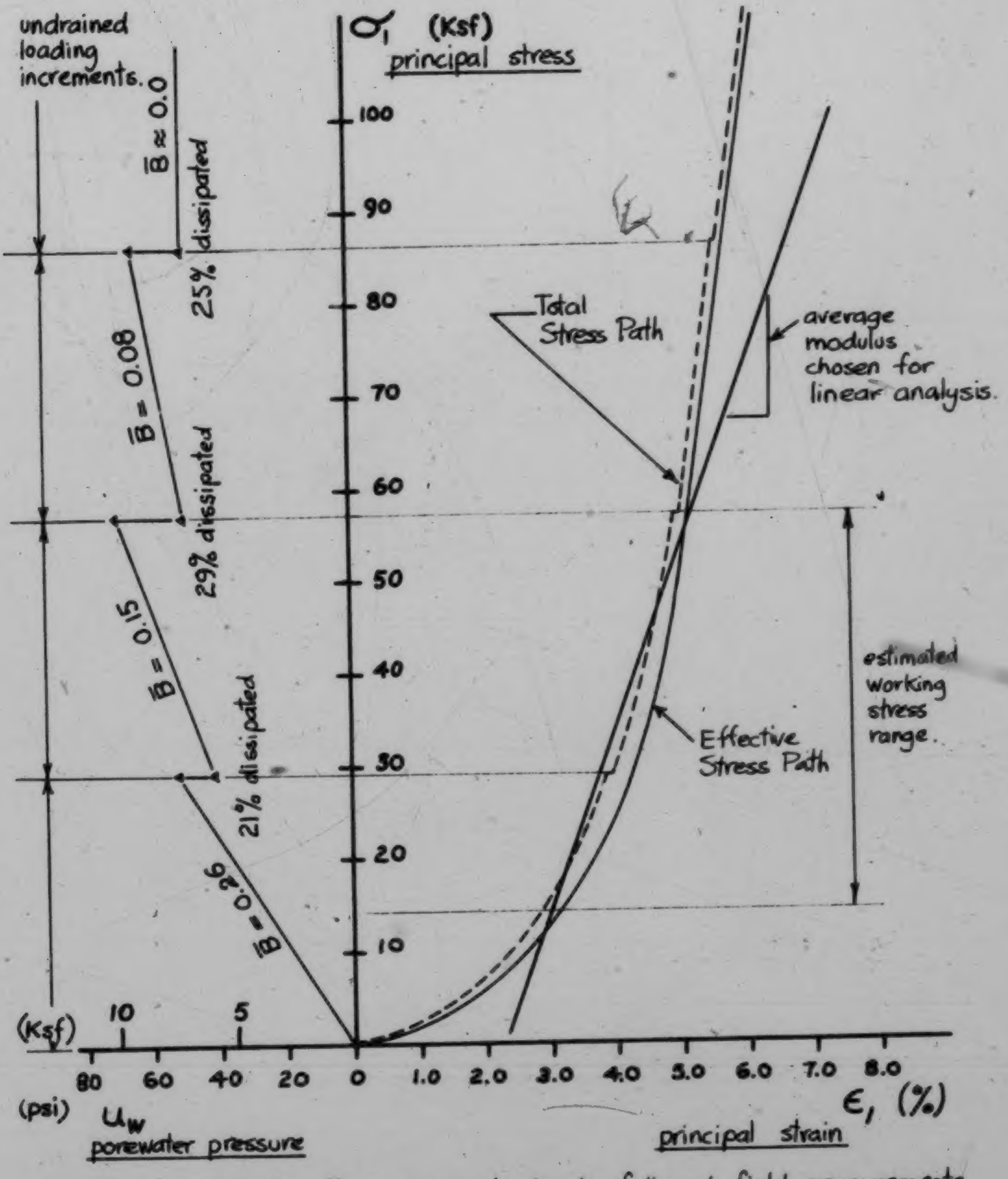
TABLE 4.3

MATERIAL PROPERTIES FOR MULTILINEAR ANALYSIS

Octahedral Normal Stress Level (Ksf)	μ	D (Ksf)	K (Ksf)	G (Ksf)
River Overburden (1)				
0.0	0.31	7800	4950	2154
0.80	0.31	8500	5385	2348
8.50	0.31	9800	6210	2708
19.20	0.31	11100	7040	3066
36.80	0.31	12300	7790	3392
48.40	0.31	13000	8250	3590
very large	0.31	13600	8620	3755
Shell Zone M2 (1) and (7)				
0.0	0.28	10140	6020	3100
4.50	0.28	10000	5930	3055
17.30	0.28	9050	5360	2766
41.00	0.28	9500	5630	2900
very large	0.28	10000	5930	3055
Core Zone M1 70-72 (3)				
0.0	0.33	1066	706	271
5.70	0.33	1695	1123	431
9.70	0.33	2220	1456	548

TABLE 4.3 (Continued)

Octahedral Normal Stress Level (Ksf)	μ	D (Ksf)	K (Ksf)	G (Ksf)
9.70	0.33	2200	1456	548
33.50	0.33	6000	3970	1523
44.50	0.33	8160	5400	2070
very large	0.33	11000	7300	2792
Shell Zone M2DI 70-71 (4)				
0.0	0.29	7350	4480	2186
very large	0.29	7350	4480	2186
Shell Zone M2DI 69 (5)				
0.0	0.29	8400	5120	2500
very large	0.29	8400	5120	2500
Core Zone M1 69 (6)				
0.0	0.38	214.3	160.0	41.7
1.67	0.37	310.0	226.0	64.2
4.70	0.36	675.0	480.0	148.3
11.70	0.36	1272	905	278
19.30	0.35	2000	1390	462
29.50	0.35	3500	2427	808
42.50	0.35	5200	3600	1200
very large	0.35	5200	3600	1200



TEST "DOT-2": standard proctor compaction, gradation 1, -¾" sizes
ω = 8.95 % γ_d = 132.1 pcf.

Of several tests, this one most closely followed field measurements.

FIGURE 4.6 CORE ZONE M1, 1969 MATERIAL:
OEDOMETER TEST (after Law, 1974)

TEST: DOT-6 kneading compaction, grad. 1, −¾" sizes, ω = 6.89% ----

DOT-14 modified proctor , " , " , ω = 6.24% ——

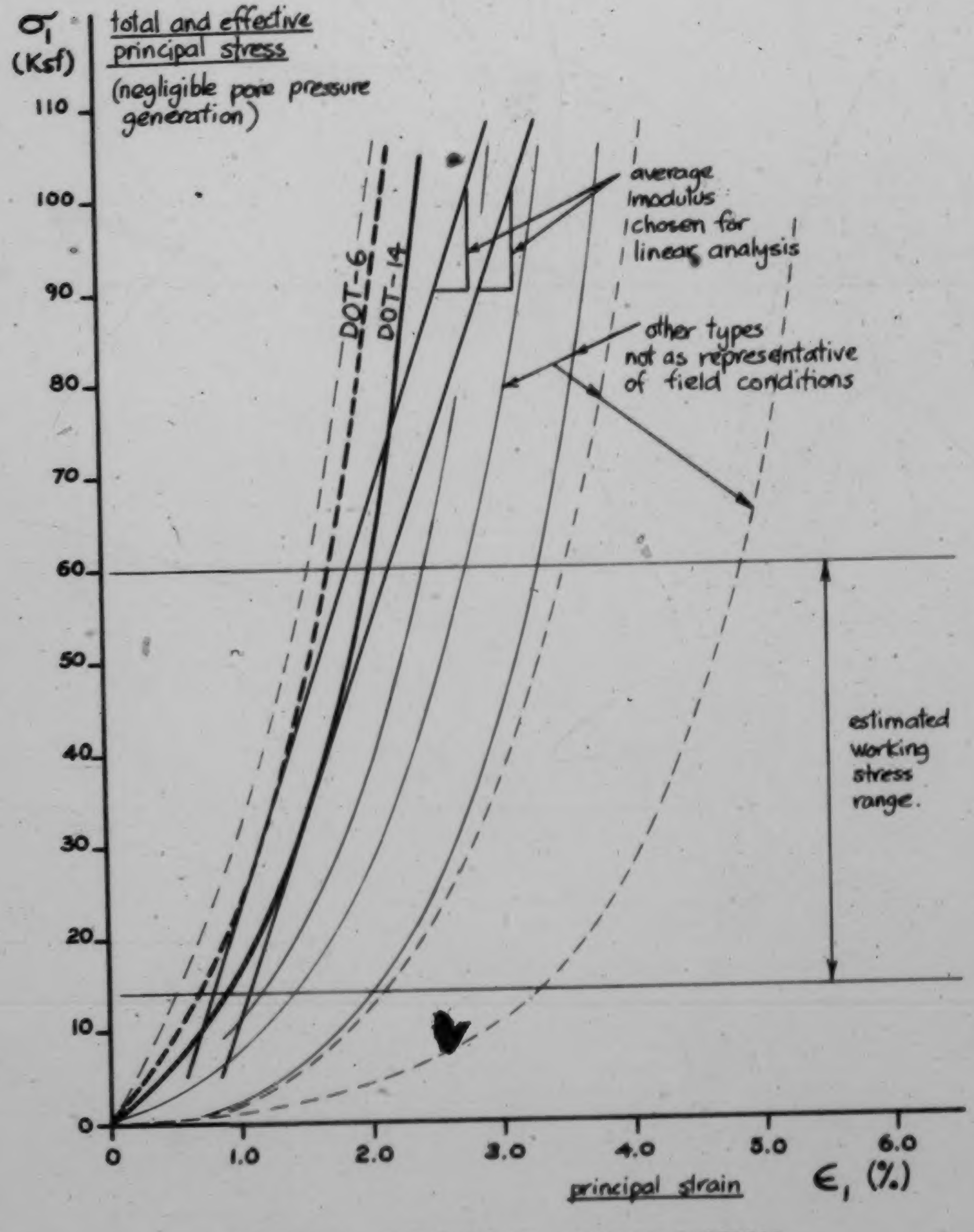


FIGURE 4.7 CORE ZONE M1, 1970-72 MATERIAL: OEDOMETER TESTS (after Law, 1974)

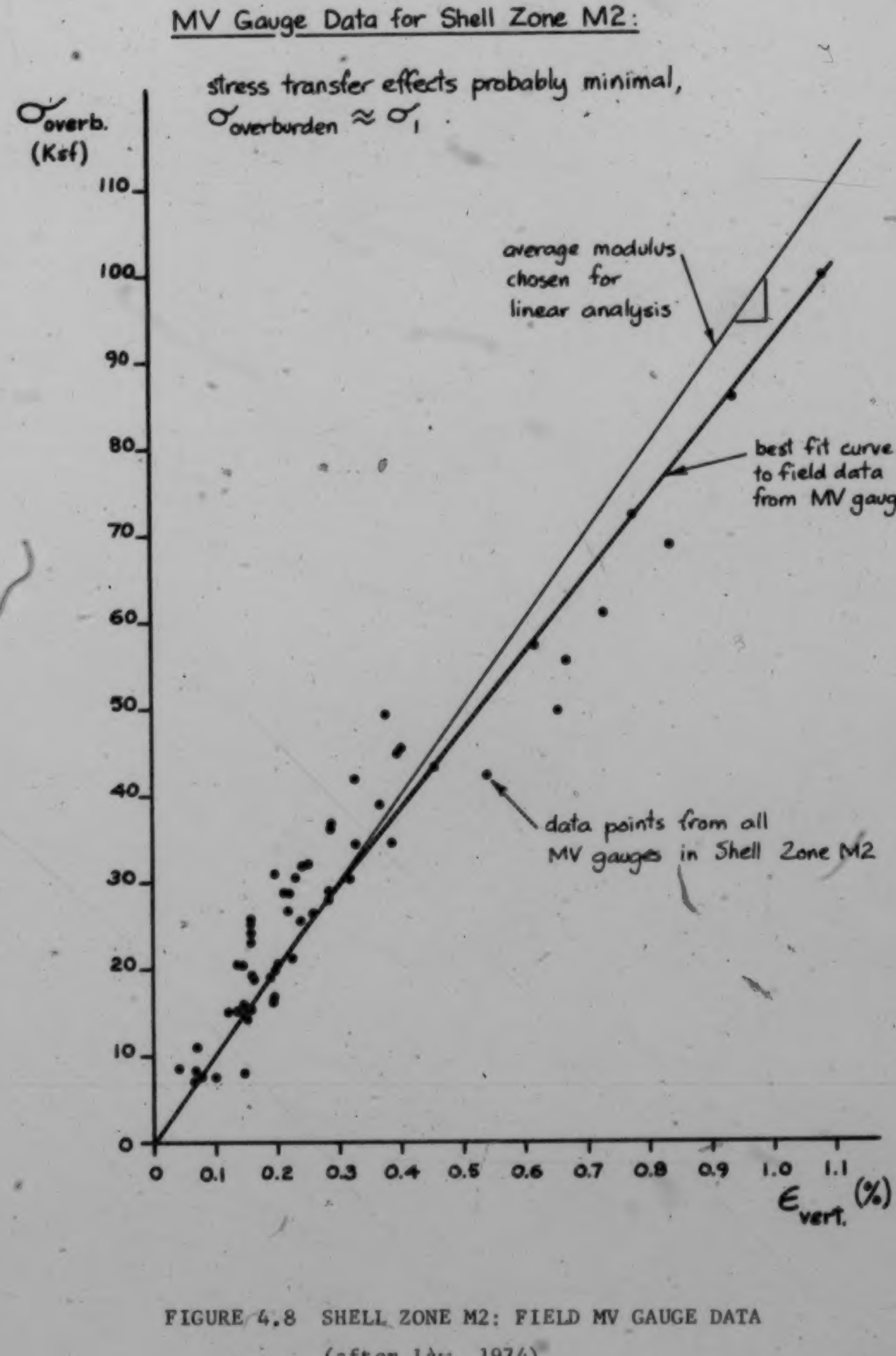


FIGURE 4.8 SHELL ZONE M2: FIELD MV GAUGE DATA
(after Law, 1974)

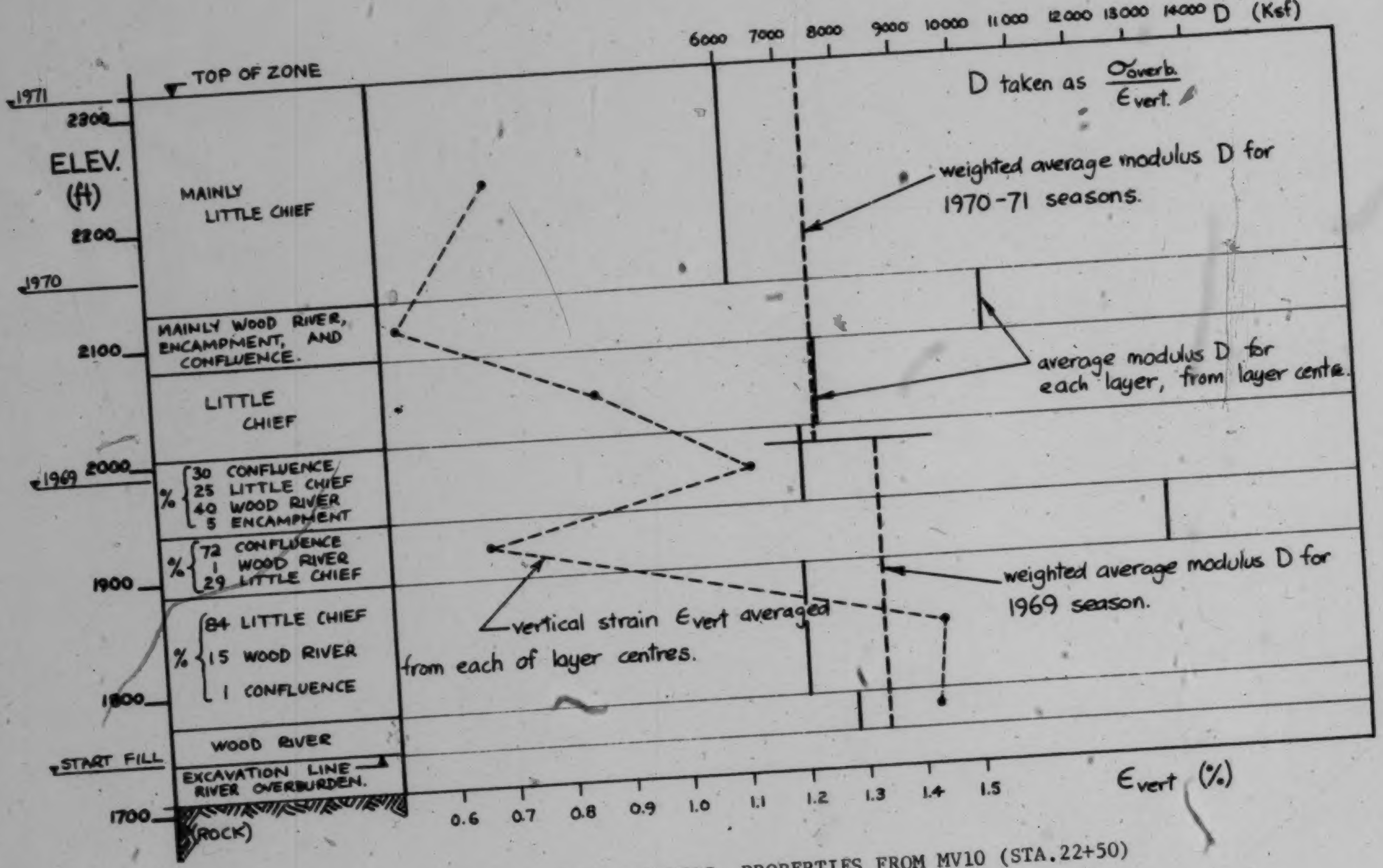


FIGURE 4.9 SHELL INNER ZONE M2DI: PROPERTIES FROM MV10 (STA.22+50) COMPARED TO MATERIAL DISTRIBUTION (STA.23+00) (CASECO)

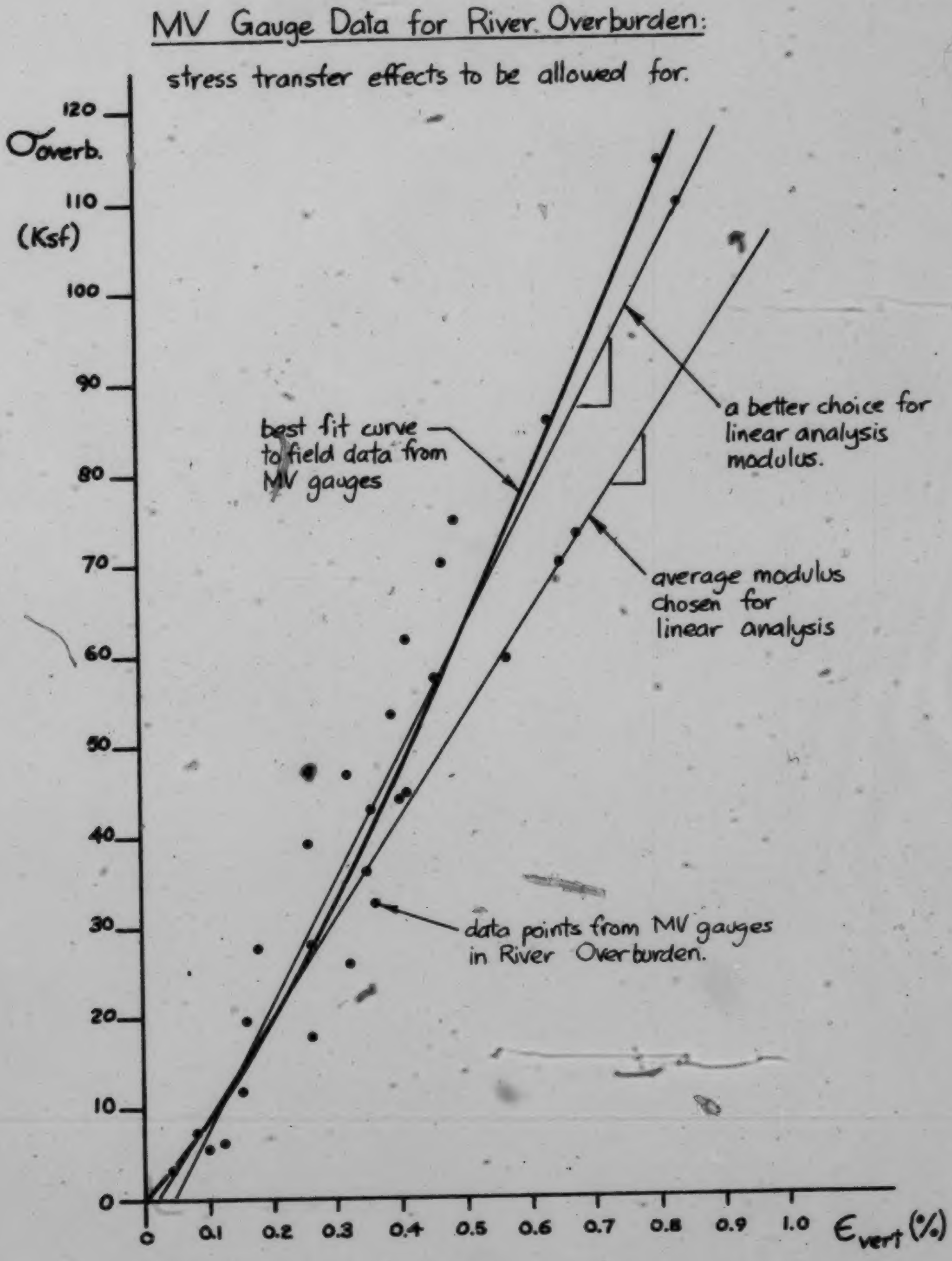


FIGURE 4.10 RIVER OVERBURDEN: FIELD MV GAUGE DATA
(after Law, 1974)

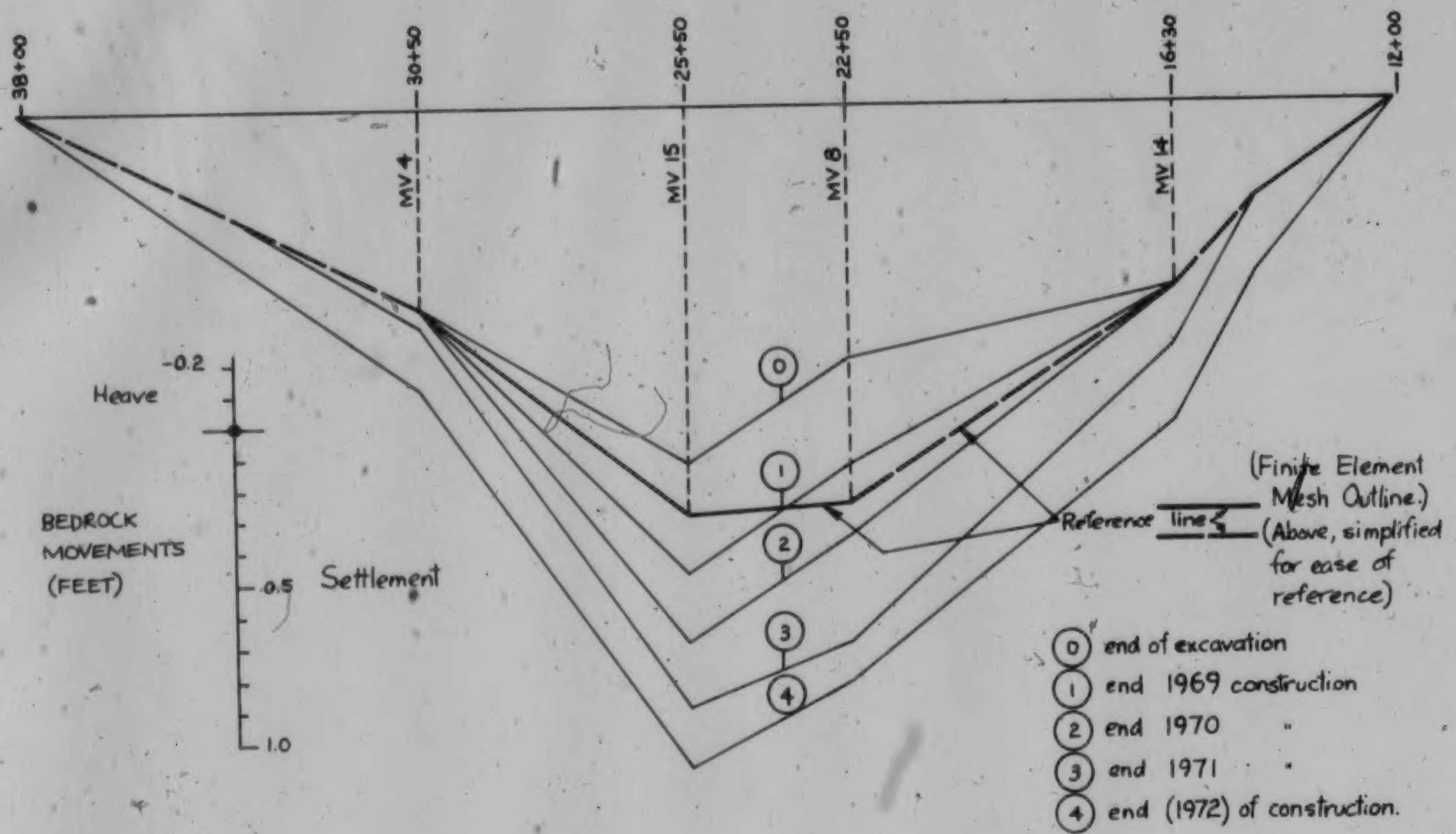


FIGURE 4.11 BEDROCK MOVEMENTS: LONGITUDINAL CENTRAL SECTION

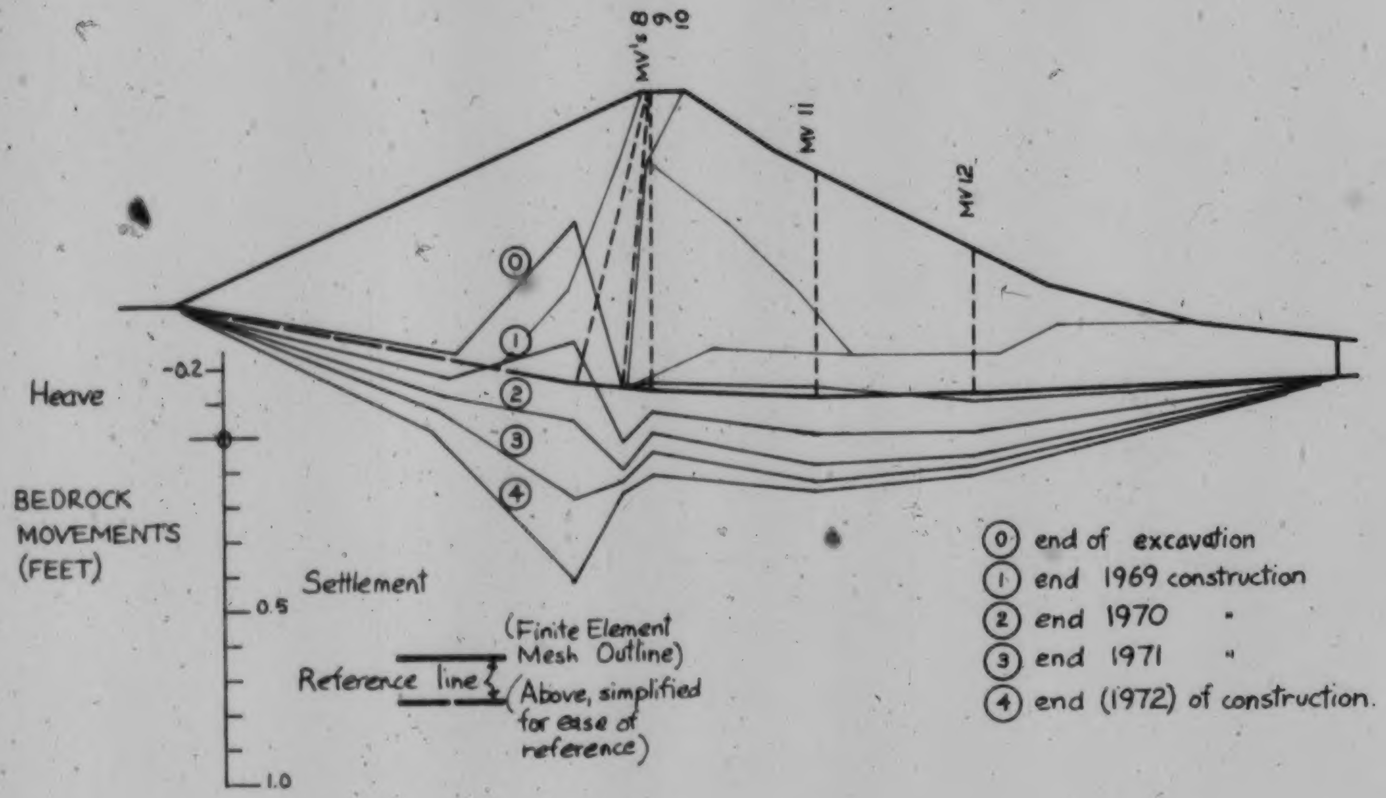


FIGURE 4.12 BEDROCK MOVEMENTS: TRANSVERSE SECTION AT STA.22+50

CHAPTER V

COMPUTATIONS USING THE THREE-DIMENSIONAL FINITE ELEMENT PROGRAM FENA.3D

5.1 Test Problems

In order to make some assessment of Krishnayya's program FENA.3D, two types of test problem were investigated:

1. A perfectly regular homogeneous dam, to help evaluate the capabilities of FENA.3D.

2. A thick column comprising some "odd" elements, to evaluate some aspects of element performance.

A wedge-shaped dam in a V-shaped valley: Data and mesh as described by Lefebvre Duncan, and Wilson (1973) were prepared and run in order to gain some experience with the program FENA.3D and to compare some results with a published work. The structure is shown in Fig. 5.1. A coarser analysis (4 lifts) was also performed. Useful information to assist in prediction of the computation time and total machine costs for further analyses was also obtained from this study.

Results for the 8-lift analysis are compared with the results of Lefebvre et al. in Appendix A. Some discrepancies occur, and may have arisen from several sources. For a further discussion of the matter the reader is directed to the notes in Appendix A. The writer concluded that with normal

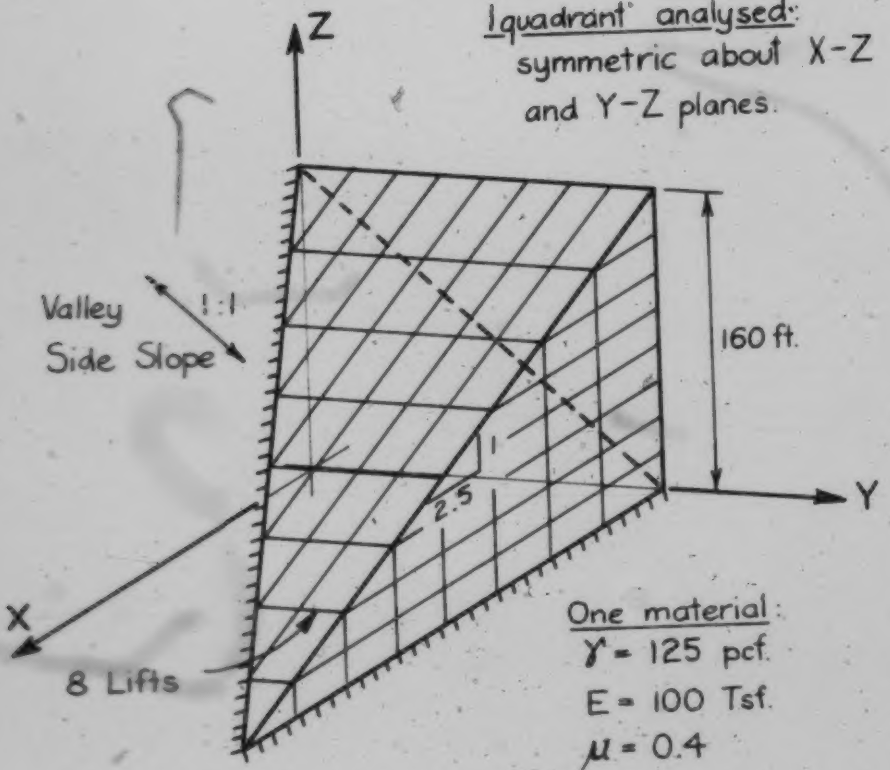

Z
1quadrant analysed:
symmetric about X-Z
and Y-Z planes.
Valley
Side Slope
1:1
160 ft.
1
2.5
Y
X
8 Lifts
One material:
γ = 125 pcf.
E = 100 Tsf.
μ = 0.4

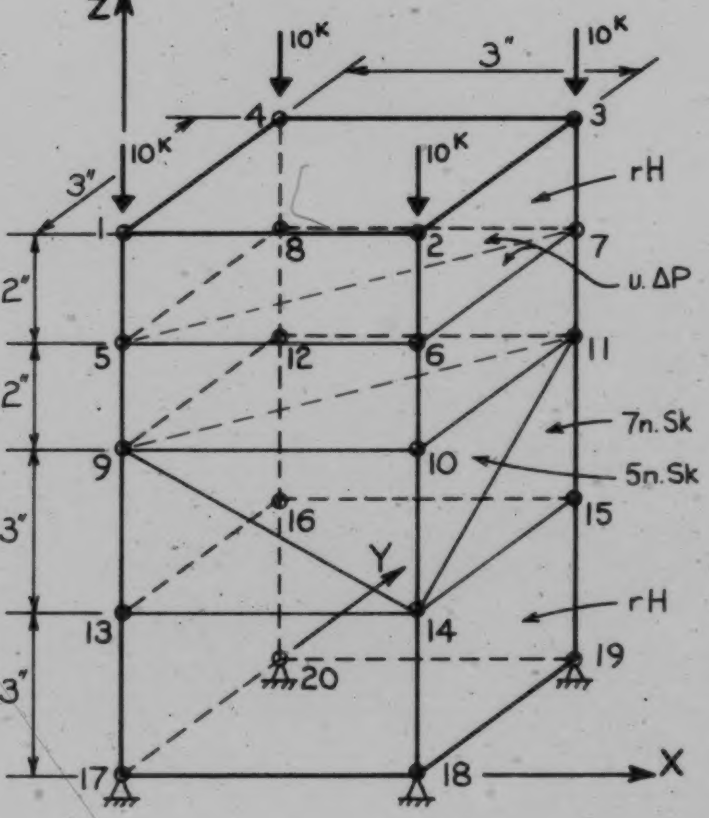
FIGURE 5.1 IDEALIZED DAM IN A 1:1 VALLEY

care in the preparation of data the results from FENA.3D would be quite satisfactory.

A simple thick column: A series of three small analyses were run on the simple steel column shown in Fig. 5.2. In theory the vertical stress should be constant at 4.444 Ksi but the effects of non-conforming elements and node-ordering of the elements was investigated. Such effects arose during construction of the Mica mesh when it was found impossible to avoid non-conforming surfaces (for example a hyperbolic paraboloid versus two triangular faces in Fig. 5.3). This physical problem has a numerical counterpart in element stiffness formulation, and even though all such situations in the Mica mesh occur in the least interesting areas, it was felt necessary to place some bounds on the inaccuracies which might accrue.

Additionally, the effect of the nodes of an element on the numerical integration procedures was examined for condensed elements. Condensed elements have one or more nodes coalesced as shown in Fig. 5.4. Though a basic order must be obeyed, the starting position can vary giving different repetition possibilities in numbering of these elements.

Results of this study are shown in Fig. 5.5 and errors of up to 22% occur. These errors are not systematic in any simple fashion and, while serious enough, the positions in the dam where the situation occurs are:


Z
10K
10K
10K
10K
3"
3"
2"
2"
3"
3"
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
rH
u. ΔP
7n. Sk
5n. Sk
rH
Y
X

E = 30 000 Ksi.

μ = 0.33

ideally, $\sigma_{vert} = \sigma_z = 4.4\dot{4}$ Ksi.

FIGURE 5.2 SIMPLE THICK STEEL COLUMN

7

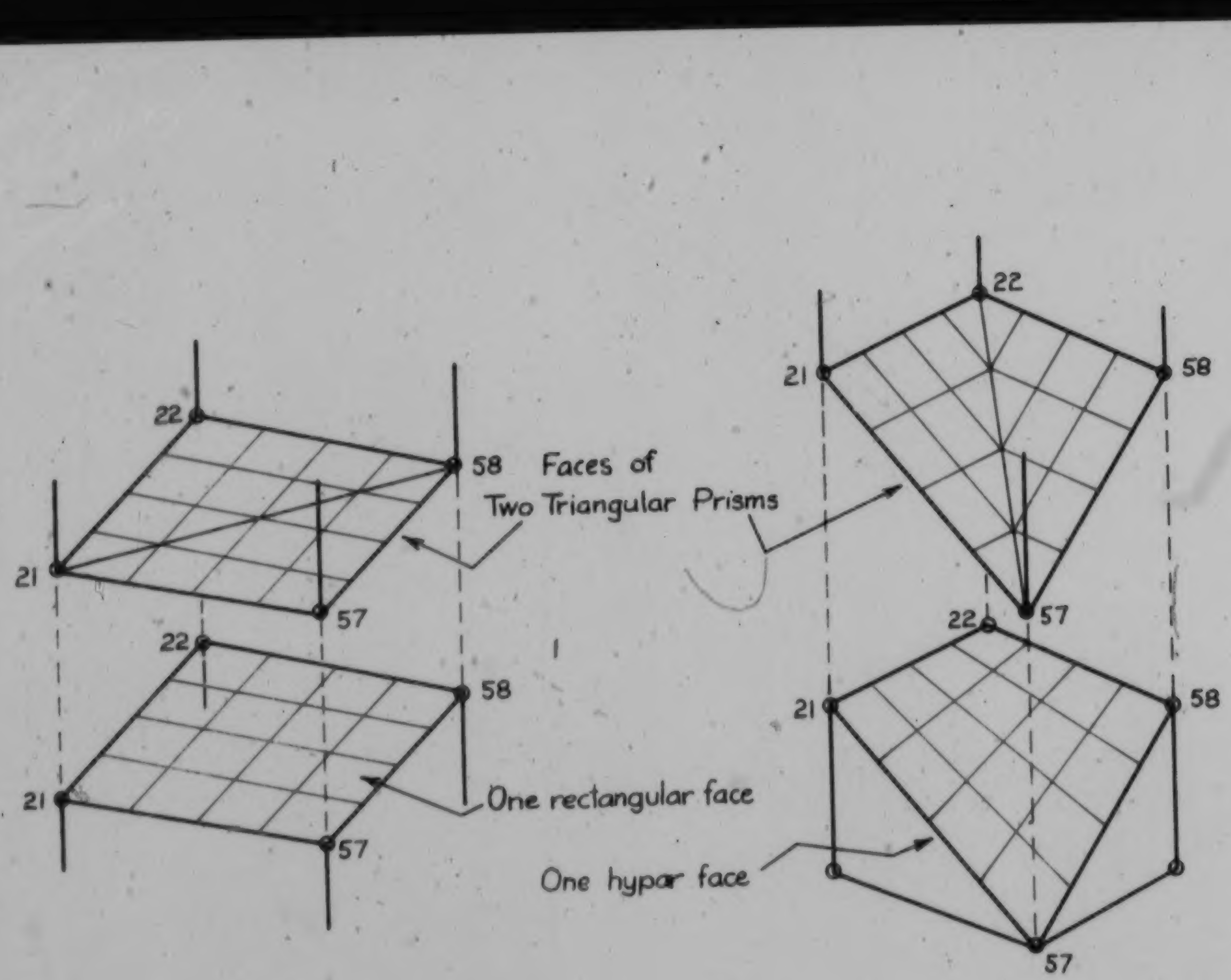
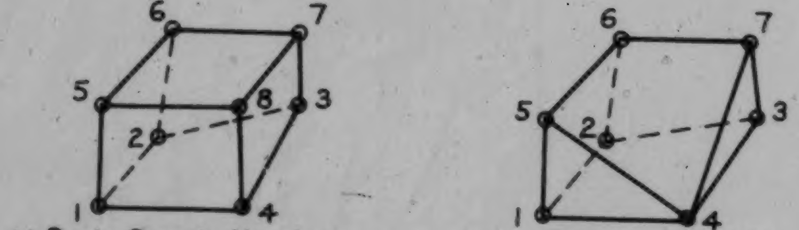
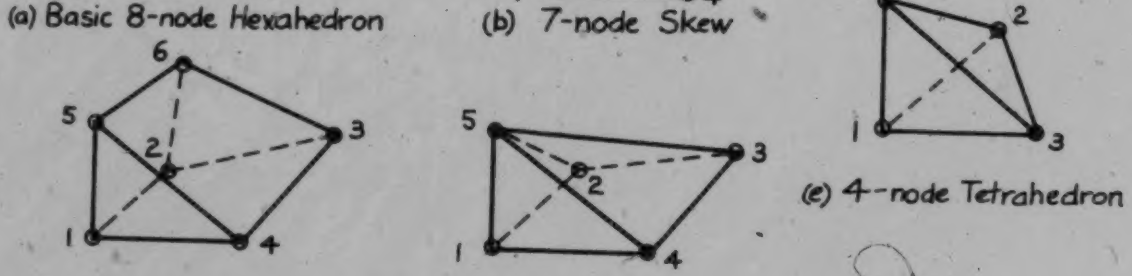

22
58
21
57
Faces of
Two Triangular Prisms
22
58
21
57
One rectangular face
22
21
58
57
22
21
58
57
One hypar face

FIGURE 5.3 EXAMPLES OF NON-CONFORMING ELEMENTS


6
7
5
8
3
2
1
4
6
7
5
2
3
1
4

(a) Basic 8-node Hexahedron
(b) 7-node Skew


4
2
1
3
6
5
2
3
1
4
5
3
2
1
4

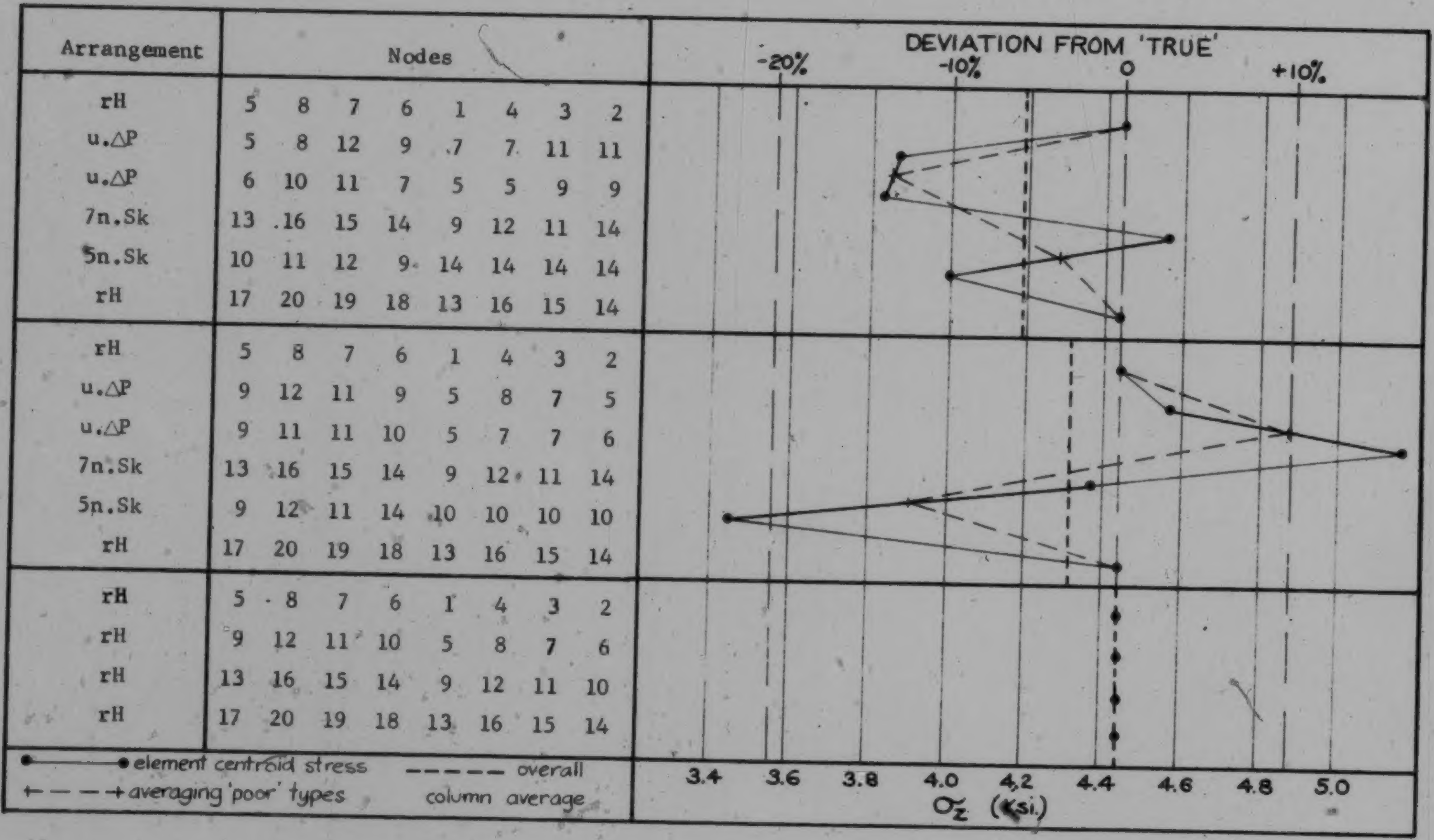
(e) 4-node Tetrahedron
(c) 6-node Prism
(d) 5-node Pyramid

Basic Numbering Scheme 1-2-3-4-5-6-7-8

Thus for (d) use : 1-2-3-4-5-5-5-5

or : 3-4-1-2-3-5-5-2 (and others...)

FIGURE 5.4 NODE ORDERING OF ELEMENTS



Arrangement	Nodes							
rH	5	8	7	6	1	4	3	2
u.ΔP	5	8	12	9	7	7	11	11
u.ΔP	6	10	11	7	5	5	9	9
7n.Sk	13	16	15	14	9	12	11	14
5n.Sk	10	11	12	9	14	14	14	14
rH	17	20	19	18	13	16	15	14
rH	5	8	7	6	1	4	3	2
u.ΔP	9	12	11	9	5	8	7	5
u.ΔP	9	11	11	10	5	7	7	6
7n.Sk	13	16	15	14	9	12	11	14
5n.Sk	9	12	11	14	10	10	10	10
rH	17	20	19	18	13	16	15	14
rH	5	8	7	6	1	4	3	2
rH	9	12	11	10	5	8	7	6
rH	13	16	15	14	9	12	11	10
rH	17	20	19	18	13	16	15	14

FIGURE 5.5 RESULTS OF COLUMN TEST PROBLEM

(a) not important from the viewpoint of the interest in stresses,

(b) liable to certain inaccuracies in any case due to artificially stiff boundary zones,

(c) unlikely to have significant influence on the structure as a whole, because of limited nodal connectivity.

Conclusions from test problems: The tests clearly indicate that in the Mica mesh, where elements are always skewed and often highly distorted and some are non-conforming and/or condensed, some numerical inaccuracies must be expected and all results accepted with certain reservations as to accuracy. The effects of numerical roundoff in the computer must be considered realistically too, the IBM 360/67 has only a 32-bit word length and all computation was done in single precision. However, consistent and reliable results will be obtained in the areas of chief interest, namely the central transverse section and the deep zones of the core.

5.2 Homogeneous Embankments

General: Kulhawy et al (1969) and Clough and Woodward (1967) have made very significant studies of homogeneous embankments as an aid to furthering the engineer's understanding of the methods and processes, and also to give a feeling for the numbers involved.

It was felt appropriate to run a series of analyses assuming that Mica Dam was homogeneous, to give a simple property (easily stored in one's head) which roughly exhibits the field deformations. Thus, a "shell" property was determined which matched settlements in the shells of the maximum transverse section (at Station 22+50), and a "core" property by matching settlements in the longitudinal core section.

"Shell" property of Mica Dam: The linear elastic parameters chosen for zone M2 were used in a 1 lift ("gravity turn on") and 5 lift analysis. The displacement pattern of the 5 lift matched field settlements well, and correlation factors for the 1 lift analysis were determined. Using similitude (with approximations) all subsequent homogeneous analyses were made in 1 lift, and the settlements converted to the 5 lift pattern. Variation of Poisson's ratio had a small effect on settlements and maximum principal stresses and a larger effect on horizontal movements and minor principal stresses, as expected from the works cited above.

To match settlements in the shell as well as possible, the following parameters were used:

$D = 9800$ Ksf $\quad \mu = 0.28$

or $K = 5810$ Ksf $\quad G = 3000$ Ksf

"Core" property of Mica Dam: It was found that core material properties derived from laboratory oedometer tests or field MV gauge data did not match but considerably overestimated core settlements. A homogeneous embankment which matched settlements in the core as well as possible had the following parameters:

$D = 4900$ Ksf $\mu = 0.35$

or $K = 3400$ Ksf $G = 1134$ Ksf

The discrepancies between these values and derived values (Section 4.5) arose because of the homogeneous structure. Further discussion of these results may be found in Section 6.1.

5.3 Analysis of the Zoned Embankment

Simple linear analysis: A 5-lift analysis, assuming rigid boundaries, was run using seven materials with properties as described in Section 4.5. Very good general agreement was obtained with the field deformation pattern. Relatively, core settlements were underestimated and shell settlements overestimated. Stresses in the core were so low that it was decided to isolate the effect of arching due to steep valley geometry if possible by using a plane strain analysis.

Plane strain analysis: The mesh between Stations 22+50 and 25+50 was isolated in plane strain by fixing all nodes on or between these sections against cross-valley movement. Surprisingly enough, the same linear elastic parameters as used above gave results very similar to the simple 3D linear analysis. Settlements were generally slightly overestimated.

Linear analysis with bedrock movements: Using the same mesh and properties as for the simple linear analysis bedrock movement during construction was simulated by applying field-measured increments at boundary nodes on bedrock, as discussed in Section 4.6. There was little change in stress distribution but a slight improvement in matching field settlements in the fill.

Multilinear analysis including bedrock movements: Using the "average stress" approach, allowing the deformation moduli in the elements to vary with stress state (as described in Section 4.5), and applying the bedrock movements, another analysis was made to determine in a simple fashion the effects of increased sophistication of material behaviour. In general stress distribution was not drastically affected and settlements mostly agreeable with field measurements.

All the results processed from these analyses, and some conclusions which can be drawn, are discussed in successive chapters of this thesis.

CHAPTER VI

RESULTS OF THE 3D MICA DAM ANALYSES

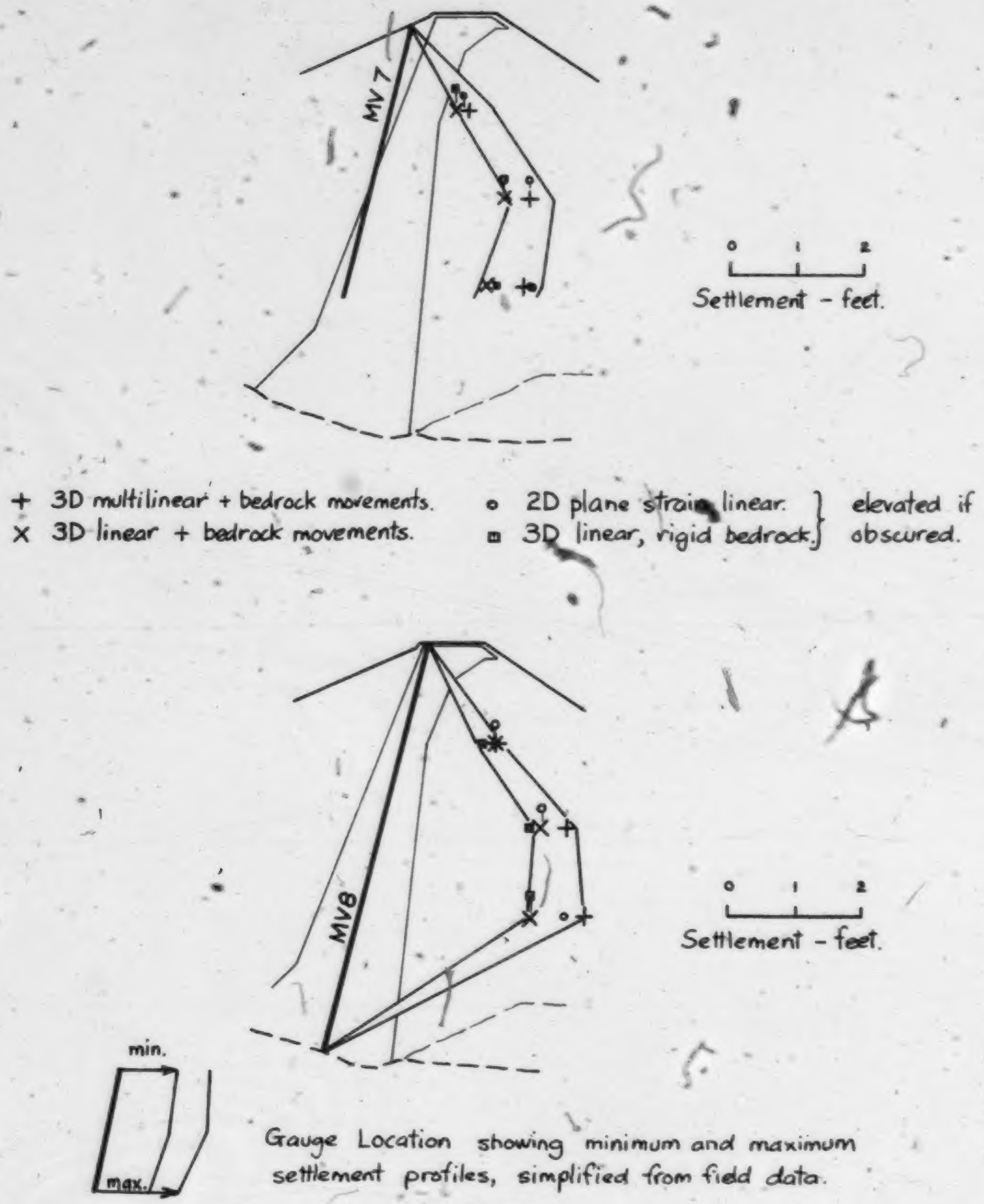
6.1 The Homogeneous Embankment Analyses

As discussed in Section 5.2, the results of the homogeneous embankment provide a guide to the sort of phenomena which occurred as Mica Dam was constructed. Zones of tensile minor principal stress were observed in the areas expected (Krishnayya, 1973a); Covarrubias, 1969). These were of limited extent and the coarseness of the mesh inhibits any more than passing interest in this. A 5 lift analysis of the homogeneous embankment did not display any tensile stresses. Examination of the displacements and normal strains in the longitudinal direction at transverse Station 22+50 definitely ruled out plane strain behaviour at this area, which would be the assumed plane strain section for 2D analyses. Regardless of this, such movements were generally about an order of magnitude less than the corresponding maximum movements and strains at this section. It is therefore concluded that, relatively speaking, an assumption of plane strain behaviour at this section is valid for 2D work, supporting the argument of Skermer (1973). Stresses calculated for the homogeneous dam are discussed underneath, and the reader is referred back to Section 5.2 for a discussion of parameters used.

6.2 The Zoned Embankment Analyses

Figures 6.1 to 6.4 show the agreement of computed settlements and field measurements for pertinent MV gauges. A field settlement plot consists of lower bound and upper bound lines. The lower bound is the cumulative settlement during construction periods only, while the upper bound is the cumulative settlement for construction and winter periods. The smaller indicates "elastic" response and the larger includes creep and consolidation effects. Plotted on the field curves (which are simplified for case of interpretation) are settlements for the cases of linear elastic 3D and multilinear elastic 3D (both with bedrock movements), simple elastic 3D and simple elastic plane strain 2D.

Since some attempt was made to include consolidation effects in the core in the modulus selection process, one would hope to see calculated core settlements tending towards the upper bound of field measurements. Since elastic parameters do not attempt to include creep effects, calculated shell settlements would preferably identify with lower bound field measurements. In general, the overall pattern of calculated settlement agrees well with field measurement. It could be argued that lack of detail in the mesh restricts core settlements and helps explain the general underestimation of field values in this zone. However, the multilinear analysis considerably overestimates core settlements in the softer (1969) material, and the explanation advanced is that

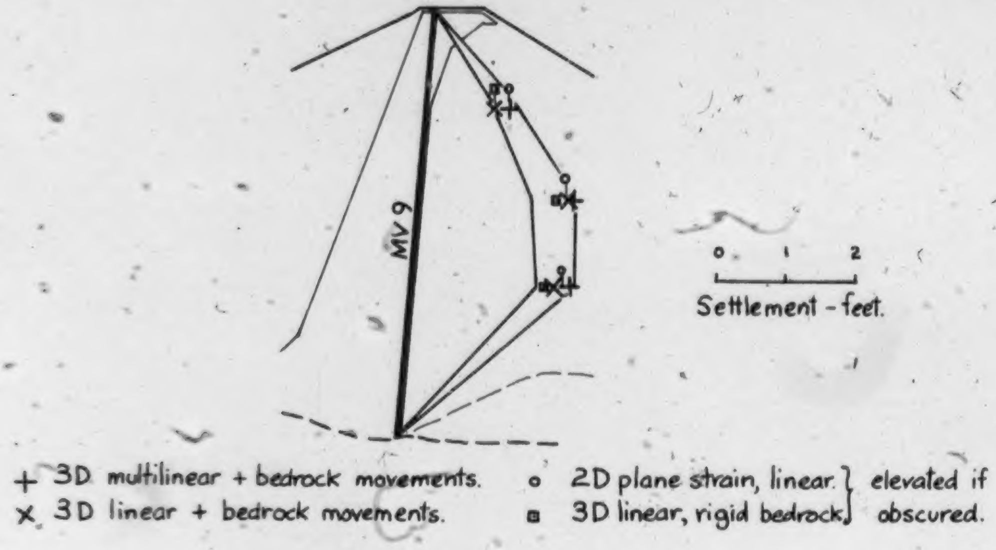

MV 7
0 1 2
Settlement - feet.
MV8
0 1 2
Settlement - feet.
min.
max.

+ 3D multilinear + bedrock movements.
× 3D linear + bedrock movements.
○ 2D plane strain linear. } elevated if
▪ 3D linear, rigid bedrock. } obscured.

Gauge Location showing minimum and maximum settlement profiles, simplified from field data.

FIGURE 6.1 SETTLEMENT PROFILES: MV7 AND MV8, STA.22+50

FIGURE 6.2 SETTLEMENT PROFILE: MV9, STA.22+50


MV 9
0 1 2
Settlement - feet.
+ 3D multilinear + bedrock movements.
× 3D linear + bedrock movements.
o 2D plane strain, linear.
▪ 3D linear, rigid bedrock.
elevated if obscured.

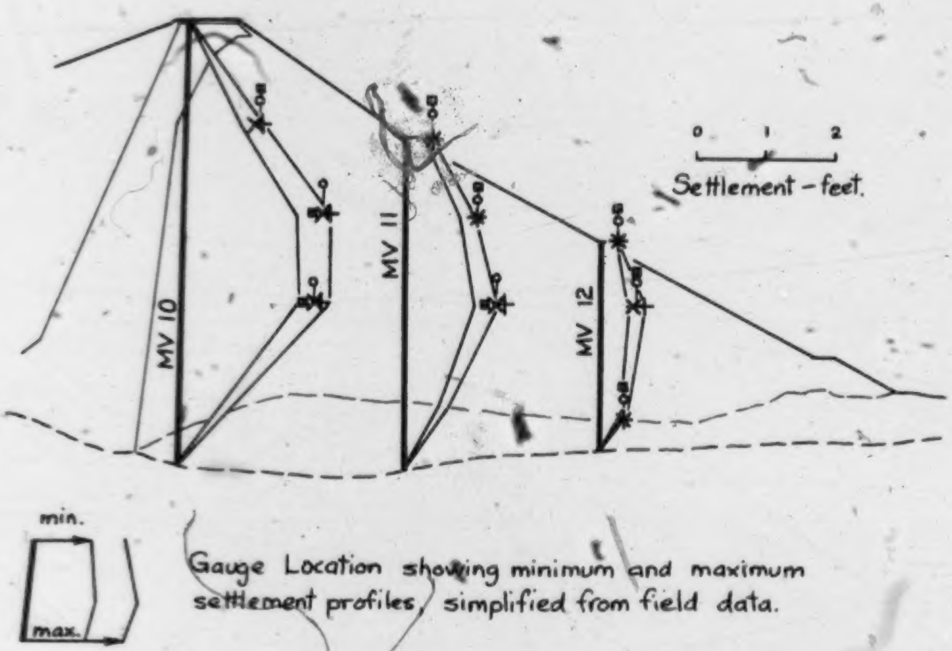

MV 10
MV 11
MV 12
0 1 2
Settlement - feet.
min.
max.
Gauge Location showing minimum and maximum settlement profiles, simplified from field data.

FIGURE 6.3 SETTLEMENT PROFILES: MV'S 10, 11, 12; STA.22+50

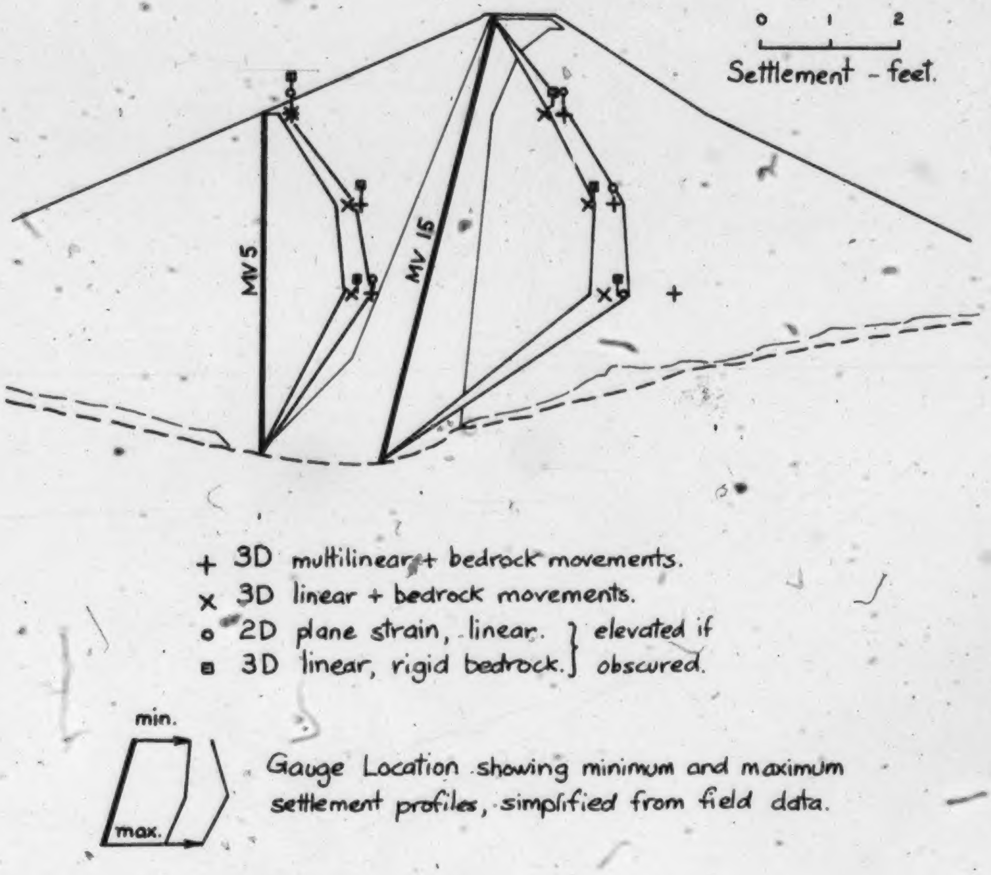

0 1 2
Settlement - feet.
MV5
MV 15
+ 3D multilinear + bedrock movements.
× 3D linear + bedrock movements.
○ 2D plane strain, linear.
■ 3D linear, rigid bedrock.
elevated if obscured.
min.
max.
Gauge Location showing minimum and maximum settlement profiles, simplified from field data.

FIGURE 6.4 SETTLEMENT PROFILES: MV5 AND MV15, STA.25+50

insufficient experience and perhaps not the best selection of test data caused this. Also, since shear failure was considered on an element basis (as described in Section 3.3, using the Mohr-Coulomb criterion) and high shear strengths are mobilized chiefly in the core-shell interface zone the finite element representation may well be too approximate. Perhaps shear failure should not have been considered in order to gain a more representative analysis with such a coarse mesh. Shell settlements were generally overestimated, and this may perhaps be blamed upon the field deformation data, which could not be separated into constructional and total strain increments. In general the settlement results are good and since settlements are good we should expect that stresses, which are less sensitive to changes in moduli alone (Clough and Woodward, 1967), should be quite realistic as compared to the structure.

The plane strain 2D settlements overestimate the 3D settlements from an analysis using identical element subdivision and material properties. This is important as it shows the effect of 3D geometry which allows deformation modes not allowed in 2D plane strain. The 2D plane strain analysis used a section of the 3D mesh. Palmerton and Lefebvre (1972) indicate that comparable 2D plane strain analyses using 2D and 3D elements show less settlement in the 2D element case. Thus a true 2D plane strain analysis which gave close agreement with field settlements may be

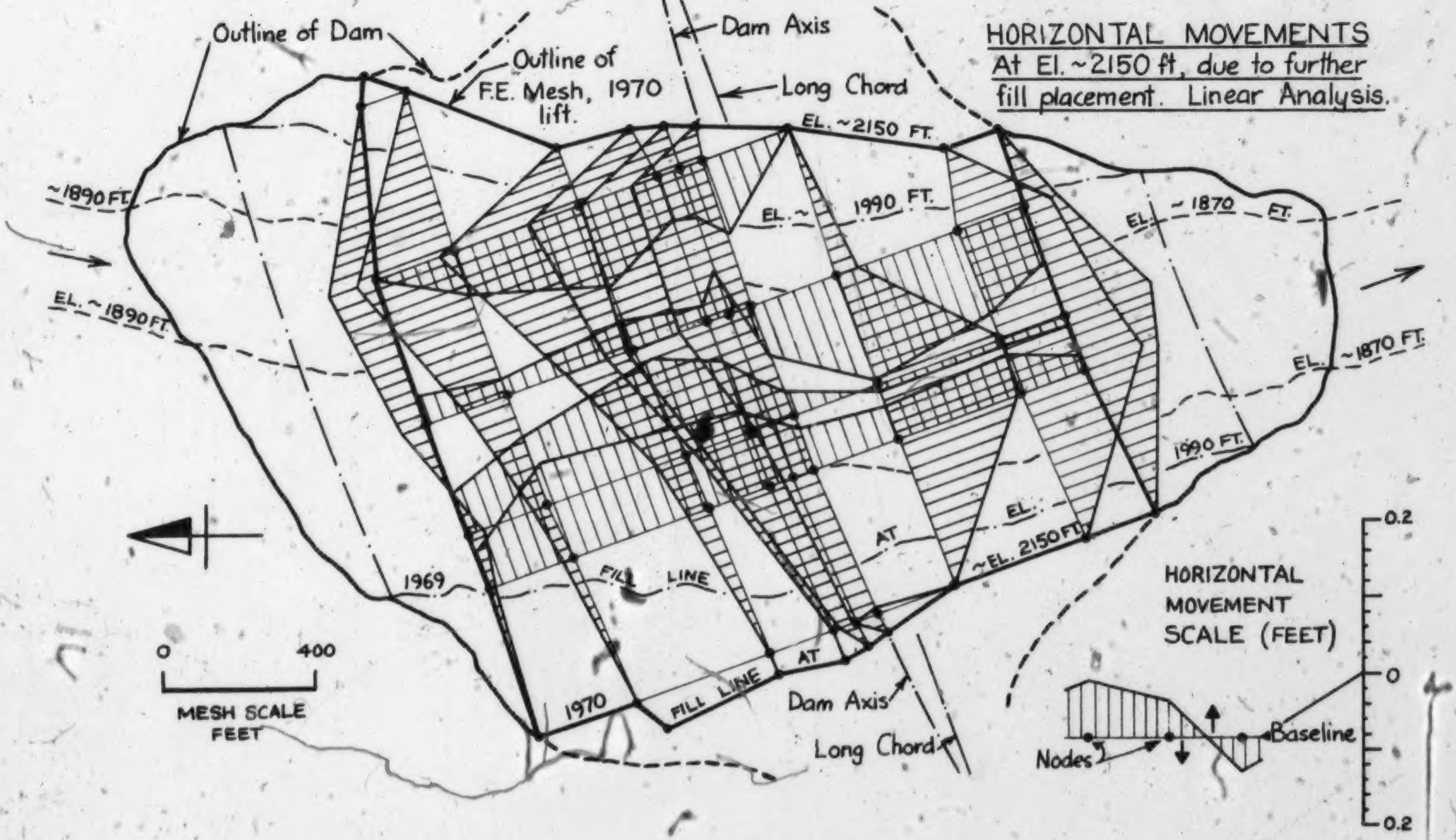

HORIZONTAL MOVEMENTS
At El. ~2150 ft, due to further fill placement. Linear Analysis.
Outline of Dam
Outline of F.E. Mesh, 1970 lift.
Dam Axis
Long Chord
EL. ~2150 FT.
~1890 FT.
EL. ~ 1990 FT.
EL. ~ 1870 FT.
EL. ~1890 FT.
EL. ~1870 FT.
1990 FT.
EL. AT ~EL. 2150 FT.
1969 FILL LINE
1970 FILL LINE AT
Dam Axis
Long Chord
0 400
MESH SCALE FEET
HORIZONTAL MOVEMENT SCALE (FEET)
0.2
0
0.2
Baseline
Nodes

FIGURE 6.5 PATTERN OF HORIZONTAL MOVEMENTS WITHIN DAM

quite representative of the field behaviour. Mica Dam was expected to demonstrate obvious 3D effects due to the changes in valley direction and slopes. That these results are not very evident may be a measure of the coarseness of the 3D analysis. Also, it may demonstrate the principle of St. Venant, the dam is a huge structure and influences from a local area are not felt at larger distances.

Horizontal movements measured in the dam are very small, and no direct comparison can be made with computed values. This is unfortunate since the role of Poisson's ratio is quite important. A pattern of horizontal movements at approx. el. 2150 ft (the end of construction in 1970) is shown in Fig. 6.5 to demonstrate typical horizontal movements expected after the embankment was complete. The lack of confirmation of the chosen values of Poisson's ratio restricts the confidence which can be placed on computed values of minor principal stresses, particularly in the core. However, the expected variation in stress would not likely be more than 10-20% of calculated values, in the writers' opinion, based upon assessments of Poisson's ratio variation in the homogeneous embankment analyses.

6.3 Stress Distributions

Core: Figure 6.6 shows a plot of element stresses for a column of core material between stations 22+50 and 25+50. There are only two elements spanning the width of the core

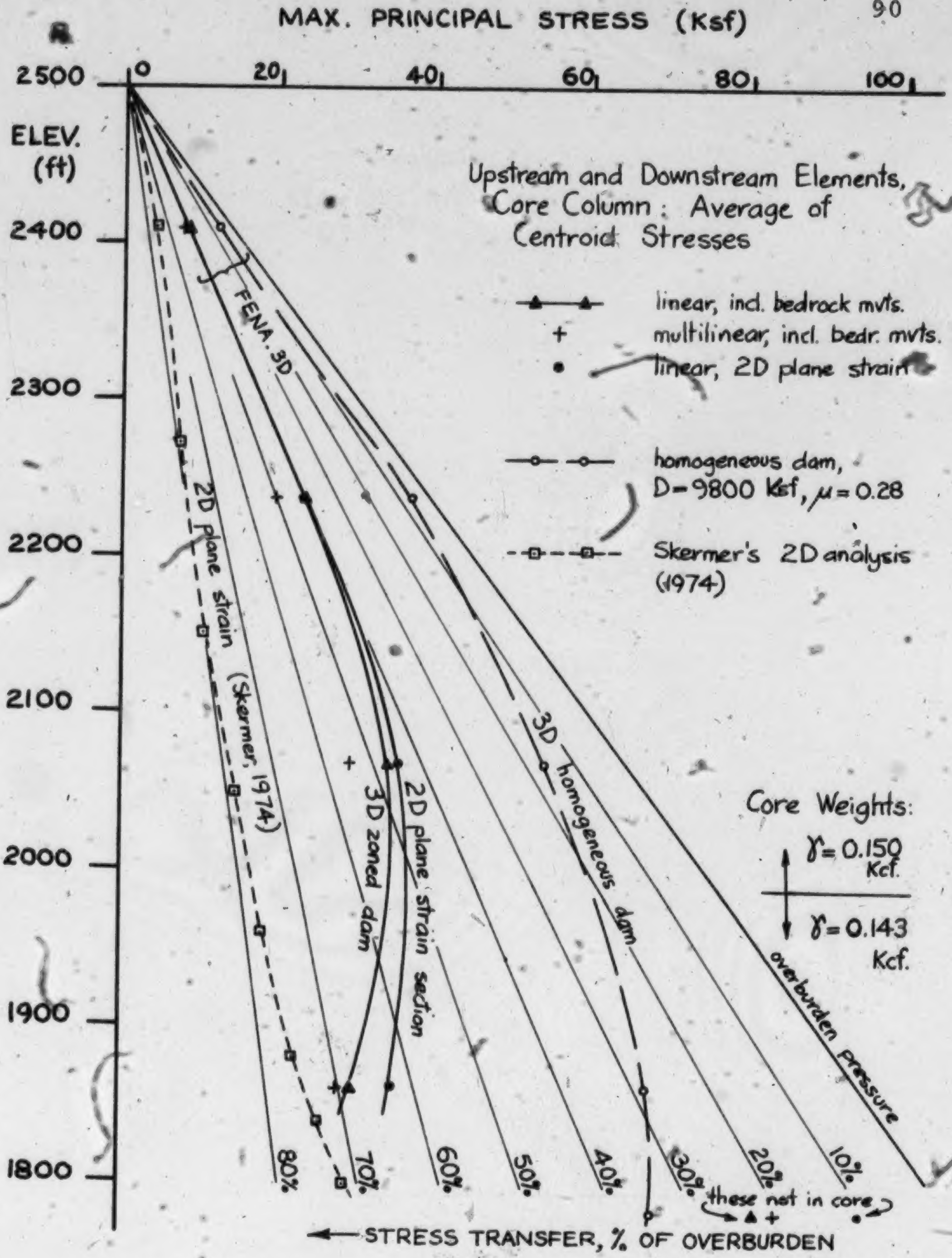


FIGURE 6.6 STRESSES IN THE CENTRAL CORE SECTION

from upstream to downstream shells, and the displacement pattern permitted is not fully representative of field behaviour. Thus it is stated clearly at this stage that the writer considers all stress results to represent average effects only, or a good first-order approximation at best.

The element major principal stresses and vertical stresses are close in value and for clarity only the principal stresses are plotted. The indication is that the prime mode of deformation is vertical settlement restrained by the mobilization of high shear strength along the core-shell interfaces, particularly the downstream side. This verifies an intuitive assessment. For the homogeneous embankment under various combinations of moduli, essentially the same vertical stress variation as that shown in the figure resulted. The conclusion is that no appreciable stress transfer occurs to about elevation 2150 ft, below which valley arching relieves about 30% of overburden stress at elevation 1860 ft. (approximately the old river level). Unfortunately a plane strain 2D analysis was not carried out using the homogeneous parameters, although the writer believes that stresses would have been very similar to those mentioned above.

In the zoned fill, however, the prime factor determining stresses in the core is the change in material stiffness from core to shell. Even in the upper reaches of the core stress transfer is about 40% of overburden; this value really starts to increase below elevation 2150 ft. with the influence of lateral (valley?) restraint, and reaches 65% at elevation 1860 ft. Somewhat surprisingly, stresses from the plane strain 2D analysis agrees very closely with 3D results. This indicates that valley restraint is minimal and that lateral restraint has only a secondary effect on stress distributions in this case.

The really significant aspect is that Mica Dam is admittedly a 3D structure, and theoretical work of previous investigators (Palmerton and Lefebvre, 1972; Lefebvre, Duncan, and Wilson, 1973) indicates that 3D effects may be quite important as regards stress distribution. The analyses described herein imply that although 3D effects may be present, the governing deformational behaviour can be represented satisfactorily using a 2D plane strain analysis. Thus, increased detail and the sophistication of stress-strain response which can be incorporated into 2D analyses should yield satisfactory results for engineering purposes, and only for a study of cracking potential would the 3D analysis be required.

A completely different, alternative conclusion is that the 3D analyses described are too coarse to provide any representative results. This conclusion is rejected by the writer until satisfactorily proven. For comparision, Skermer's (1974) stress results using a 2D analysis are also shown on Fig. 6.6.

Shell: The stresses computed for the shell are not much different from what one might expect. The upstream shell essentially sits tightly in place over the core, and the core widening at the base causes high stress levels as the shell overburden is thrown into a smaller bearing area. The downstream shell essentially sits tightly in place, partially under the core, and participates more strongly in core-shell interaction than the upstream shell. The weaker zone M2DI exhibits no appreciable stress transfer at depth because it holds the burden of core-shell interaction, and also because of the wedging action of the foundation excavation. The shell stresses for the linear analysis including bedrock movements are shown in Figs. 6.7 to 6.10. It should be noted that these stresses are quite comparable with Skermer's results, although he did not model material differences to the degree adopted in the 3D work.[1]

Overall stress distribution: For the sake of comparative evaluation stress distributions in transverse and longitudinal sections are given in some detail for various analyses. They are all grouped together for convenience in Appendix D. Data is plotted using engineering judgement and without making any allowance for artifically stiff boundary effects. Because of the coarseness of the mesh there is considerable scope for personal interpretation of contour patterns, although the writer has endeavoured to be consistent.

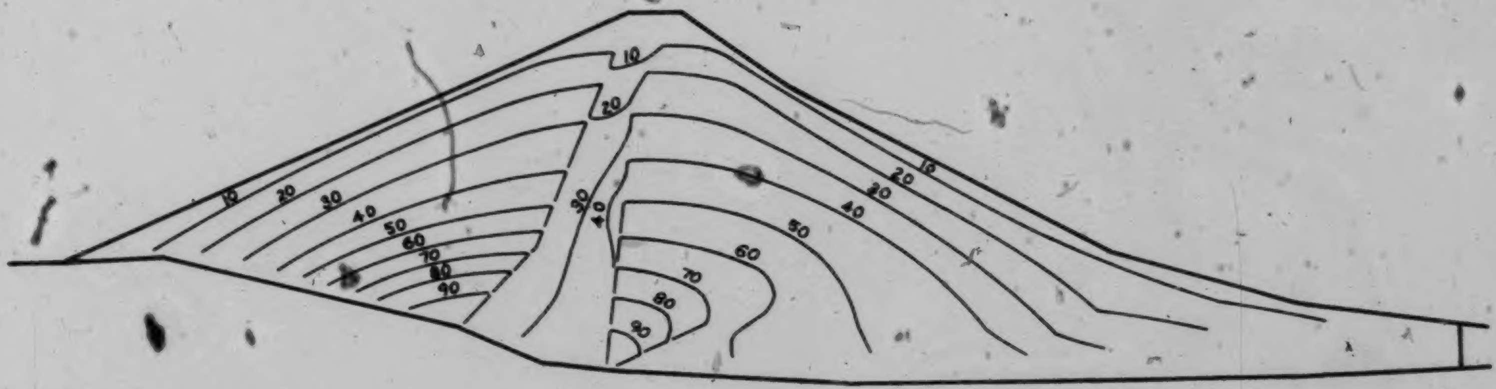


FIGURE 6.7 TRANSVERSE SECTION, LINEAR ANALYSIS: MAJOR PRINCIPAL STRESS CONTOURS, IN KSF.

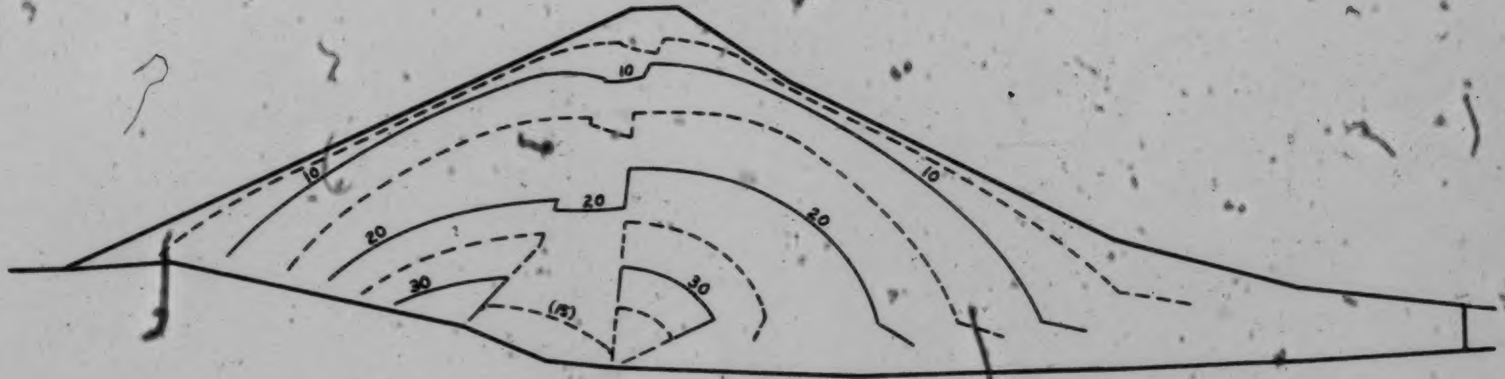

10
10
10
20
20
20
30
30
(15)

FIGURE 6.8 TRANSVERSE SECTION, LINEAR ANALYSIS: INTERMEDIATE PRINCIPAL STRESS CONTOURS, IN KSF.

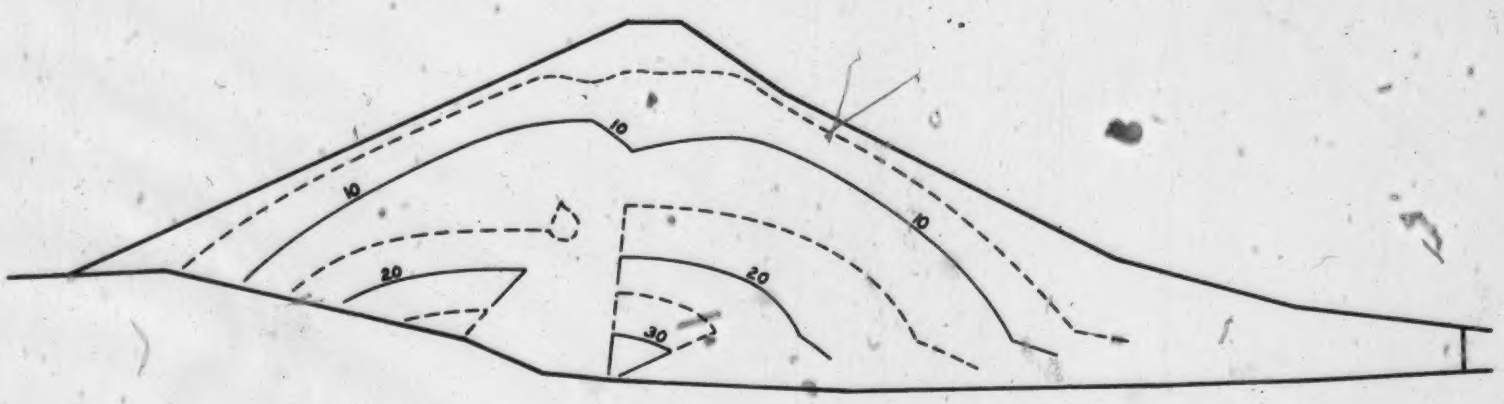

10
10
10
20
20
30

FIGURE 6.9 TRANSVERSE SECTION, LINEAR ANALYSIS: MINOR PRINCIPAL STRESS CONTOURS, IN KSF.

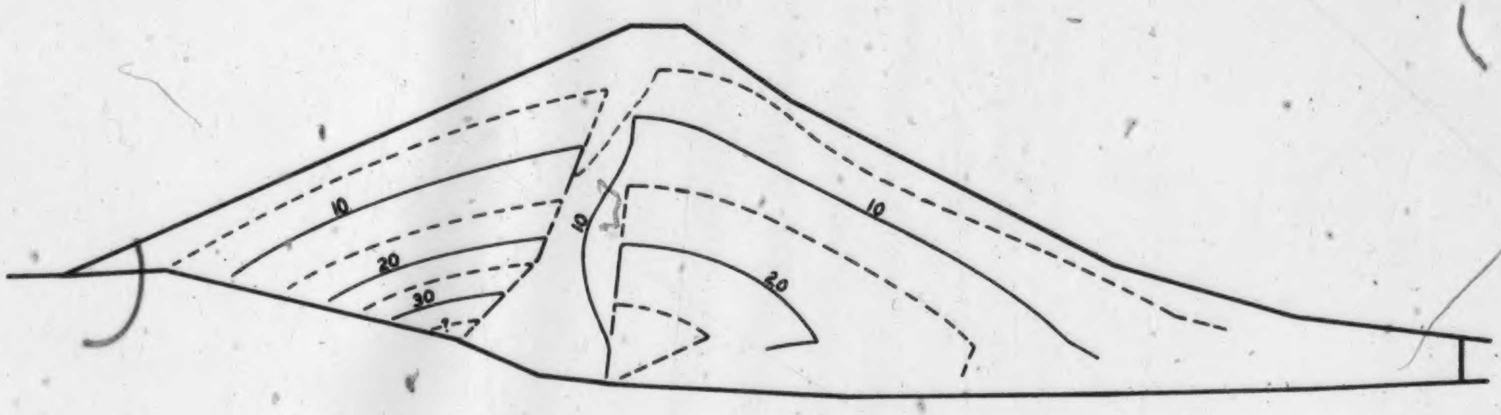

10
20
30
10
10
20

FIGURE 6.12 TRANSVERSE SECTION, LINEAR ANALYSIS:
MAXIMUM SHEAR STRESS CONTOURS, IN KSF.

Element stresses are calculated at the element centroid and nodal averaging procedures were not used because averaging the values of such a coarse mesh was found to be unsatisfactory. Stresses in the transverse section shown as Station 22+50 are actually in the elements between Stations 22+50 and 25+50. Stresses in the longitudinal core section are averaged from the upstream and downstream core elements.

Attention is drawn to the low stresses in the core, a serious situation which may lead to hydraulic fracture. This is described further in Section 7.2. As an estimate the writer believes that stresses at some point (particularly in the core) would not vary by more than 10 to 20% from the results of his anslyses. The stress situation in the core of the dam is not as satisfactory as it could be, that is the simple truth of the matter. Whether there is any practical significance in this point is a matter for ongoing research and debate.

6.4 Stress Paths of Elements

The elastic parameters that were used to represent deformational behaviour at varying stress levels only crudely represent the physical processes of the material. Some insight into the approximations dealt with can be found by plotting an element stress-strain path on a background of (say) oedometer tests for that material. Also, the plot of octahedral normal stress versus maximum principal stress may yield some insight into the behaviour represented by Poisson's ratio. Plots for a typical core element are shown against oedometer data for the core material, for the linear analysis which gave the best agreement with field deformations, on Fig. 6.11. On Fig. 6.12 are octahedral stress/principal stress plots for several core elements. Rough correspondence of moduli are shown, and the effective value of Poisson's ratio is close to constant at the assumed value. The same is basically true of shell zone M2 although the effective value of Poisson's ratio drops off a little with increased stress level. (Figs. 6.13 and 6.14).

The conclusions which can be drawn from this are limited. One would expect that the material might exhibit a wider variation in Poisson's ratio, but that some average value may be broadly representative. One would also expect that materials might exhibit considerable variation in moduli, but that some average modulus could describe this adequately. It is not absolute values of elastic parameters which govern stress distribution but rather the ratio of their values.

6.5 Comments on Some Important Influential Factors

Shear failure of elements: In accordance with the work of Kulhawy et al (1969), elements in which computed shear stresses exceed the calculated failure shear strength (using the Mohr - Coulomb failure criterion) are assigned

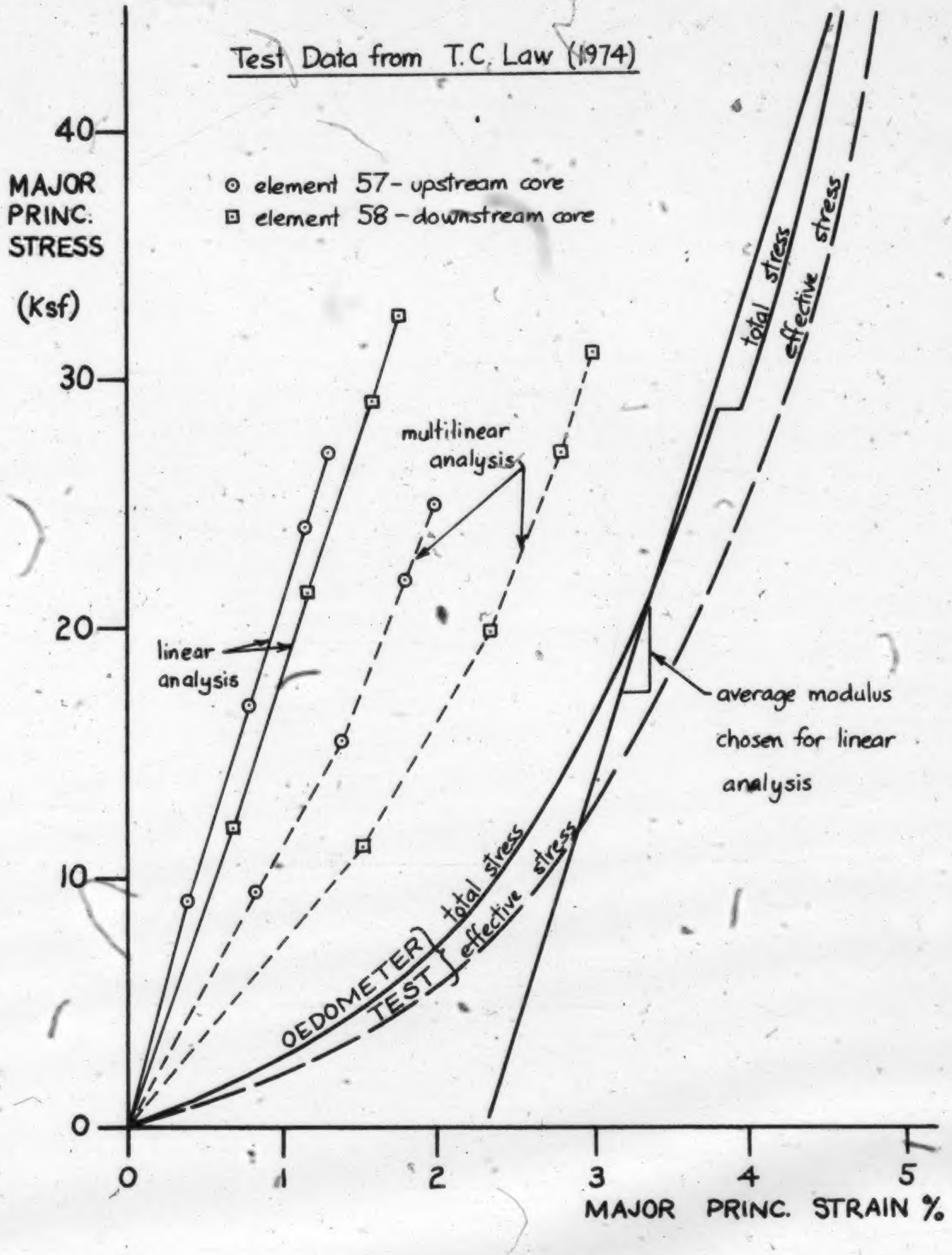


FIGURE 6.11 STRESS PATH: 1969 CORE ZONE M1 ELEMENTS

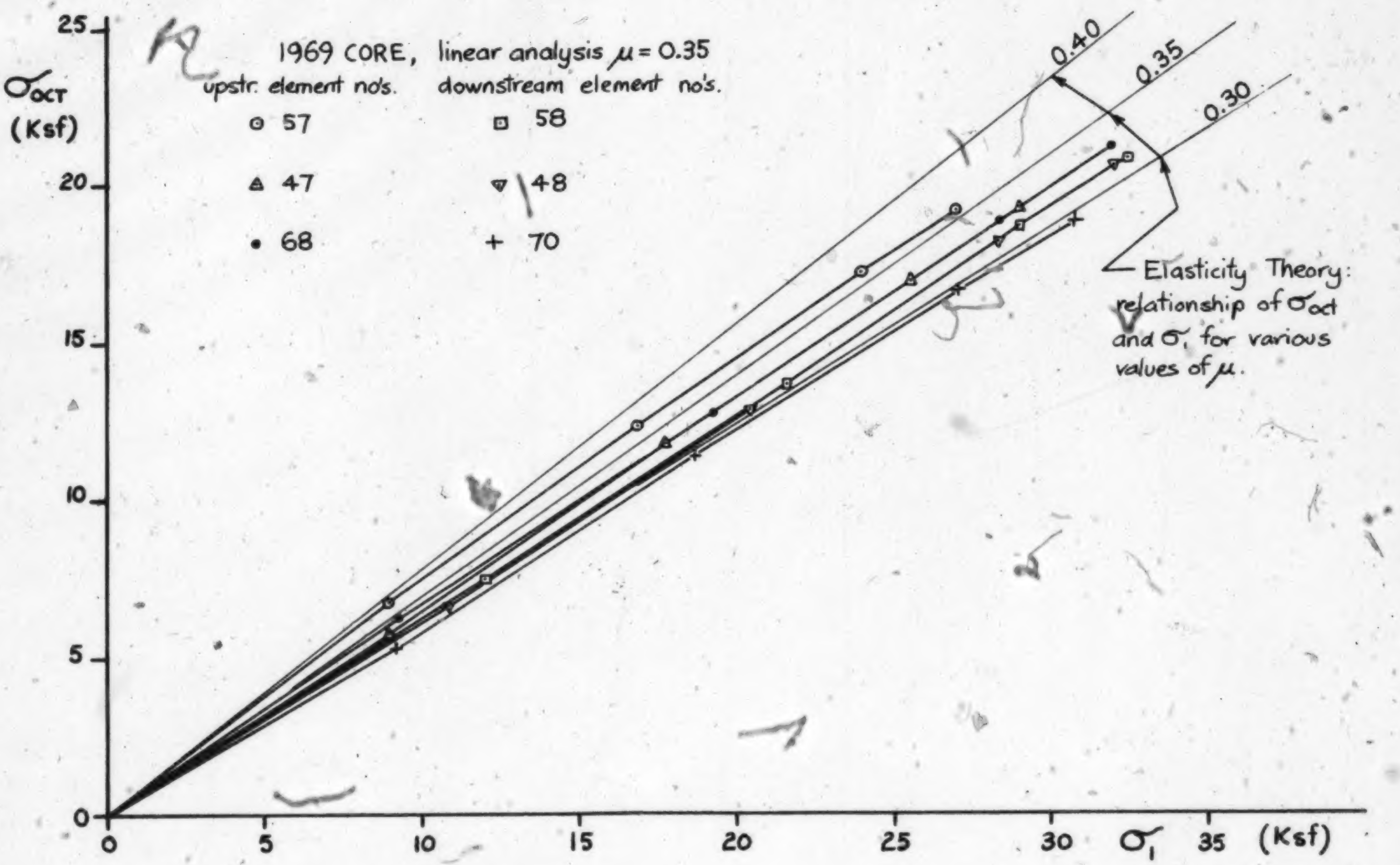


FIGURE 6.12 STRESS INTERRELATIONSHIP: 1969 CORE ZONE M1 ELEMENTS

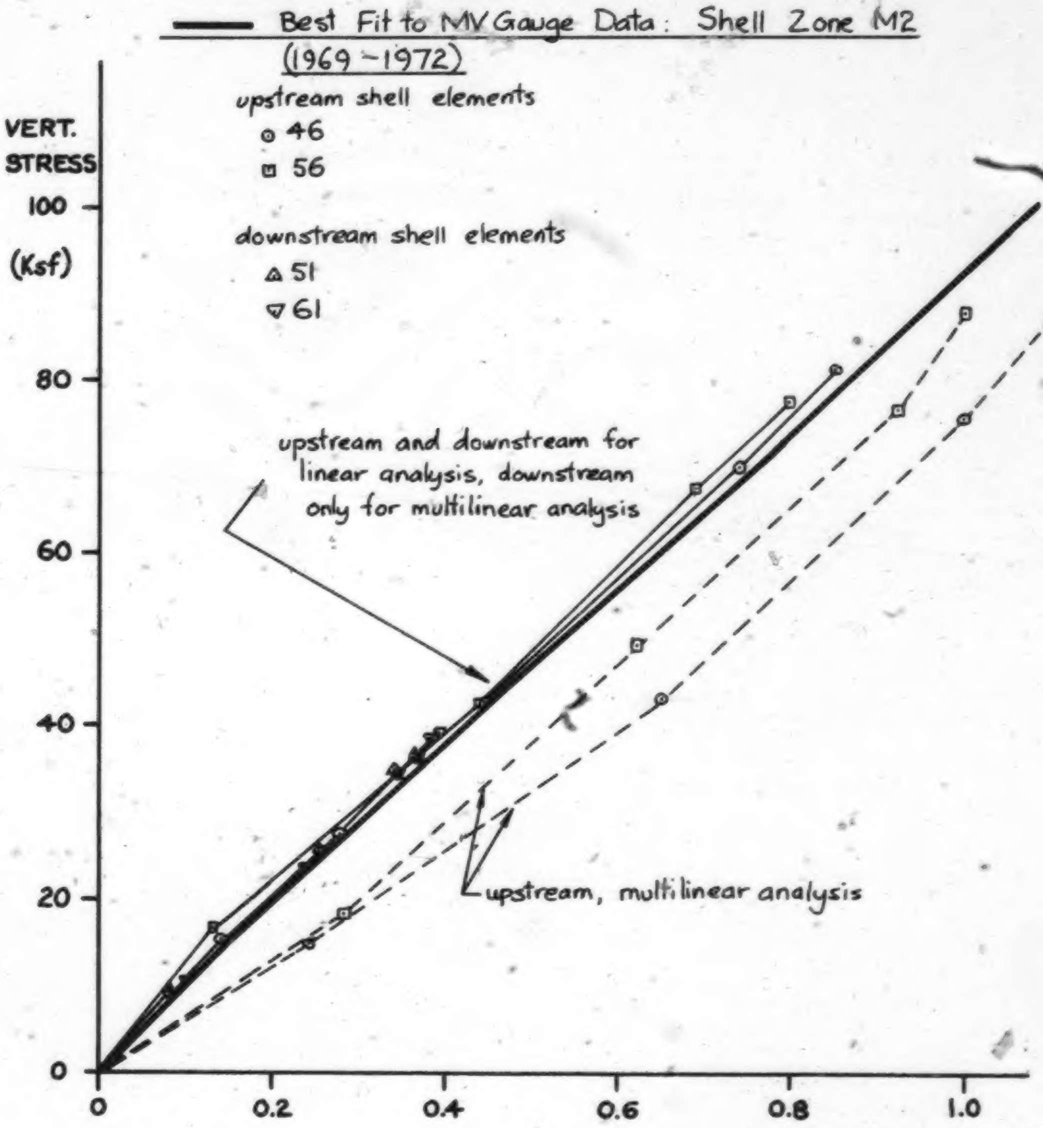


FIGURE 6.13 STRESS PATH: SHELL ZONE M2 ELEMENTS

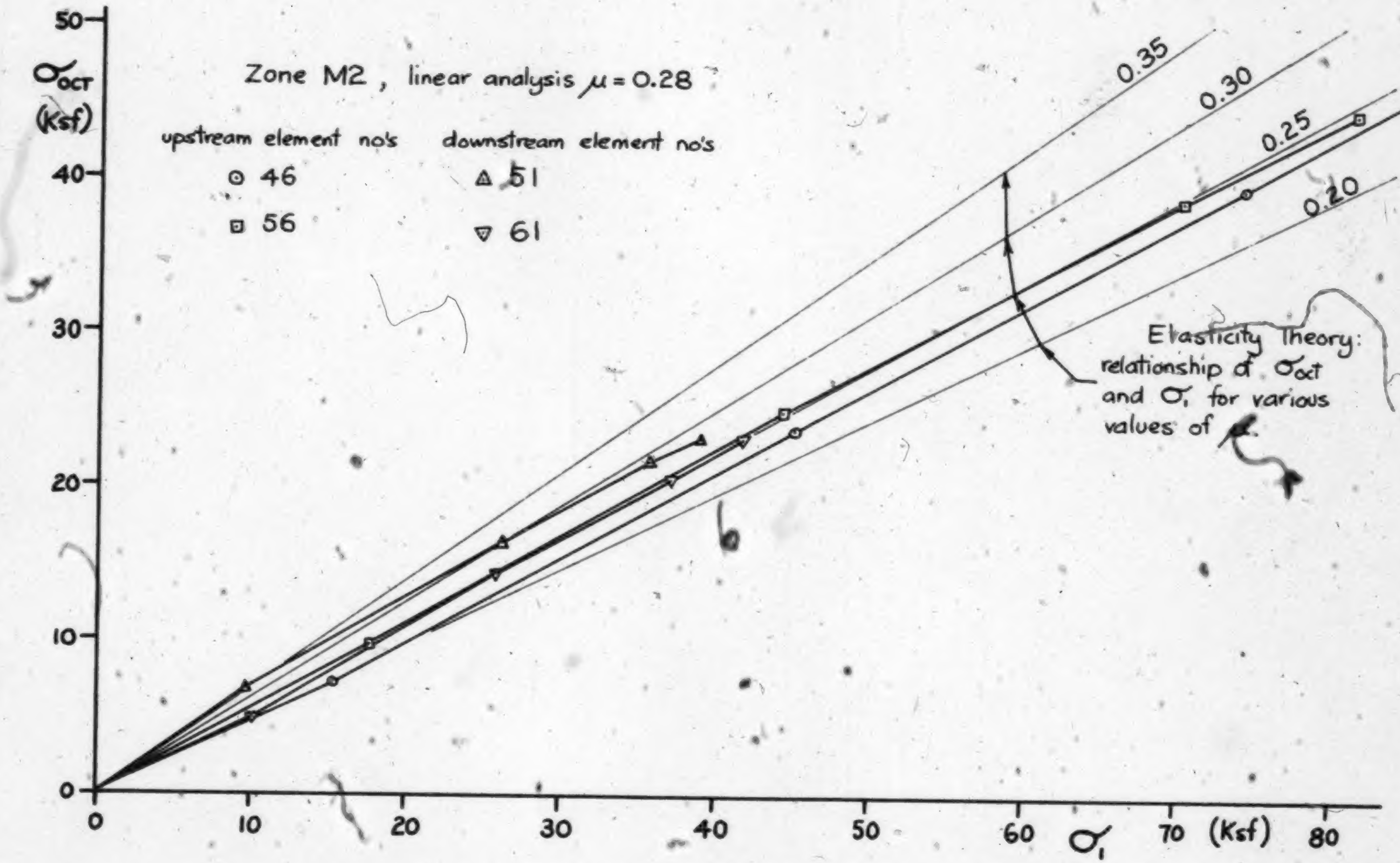


FIGURE 6.14 STRESS INTERRELATIONSHIP: SHELL ZONE M2 ELEMENTS

low values of shear modulus for subsequent stiffness calculations. In principle this feature is worthy of inclusion, but there is some doubt in the writer's mind as to its suitability in a coarse mesh, as Krishnayya (1973a) showed that failed zones are governed to some extent by mesh size. In these analyses too much material may have been assigned "failed" properties, although preliminary estimates indicated that the "failure - no failure" option would have less than 10% influence on stress distribution. Elements which failed in shear were either highly distorted elements in poorer and unimportant areas of the mesh, or elements at abrupt changes of slope in the longitudinal core section.

Ideally an analysis which considers shear failure and which also uses the "no tension" approach (Zienkiewicz et al, 1968) would be desirable. The writer doubts whether either is practical or realistic in the present analyses.

Sophistication of stress-strain relations required: The evident success of the finite element method as applied to fill construction analysis strongly suggests that, for the stress paths considered, representative behaviour can be obtained by simple approaches to nonlinearity, or representative linear parameters which have to be chosen very carefully. This is an encouraging result for it makes the method much easier to use in practice, and allows efforts to be concentrated upon proper simplifications for future research.

No one should ever believe the results of a finite element analysis verbatim without careful assessment of the considerations which went into choice of parameters. This work on Mica Dam reinforces the opinion that finite element analysis is now a useful design tool in geotechnical engineering, when used wisely.

General accuracy required and claimed: Inaccuracies of many types were bound to accrue in the Mica Analyses. Coarseness of the mesh, element performance, representative stress-strain behaviour, options for shear failure, and numerical roundoff all are present. No serious probabilistic approach to assessment of overall errors is possible. The maximum likely error is estimated as about 15-20% and it is claimed that it would not be justifiable to seek a reduction of this figure. Furthermore, these results are quite acceptable for practical soils engineering purposes. To seek greater accuracy would be unrealistic and contrary to well established practices. Nothing should ever supplant the observational method as the mainstay of soil engineering practice.

Cost of 3D analyses. Some remarks concerning the costs involved in making the analyses may be found in Appendix E.

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[1] Skermer (1974) used only three linear elastic materials in a rather arbitrary fashion, and his results must be treated with caution.

CHAPTER VII

SOME IMPLICATIONS OF THE STRESS ANALYSES

The end of construction showed that the Dam was performing satisfactorily. Embankment movements were smaller than anticipated and the structure was deemed successful. One of the consequences of these small movements, perhaps, is the exceptionally low stresses in the core due to the high degree of stress transfer between core and shells. In the light of these low stresses the following questions arise:

(1) What effect will the reservoir have, on first filling and in the long term?

(2) What possibility is there of hydraulic fracturing of the core?

(3) Do these low stresses reflect any concern in the safety of the structure with respect to earthquake loadings?

7.1 First Filling of the Reservoir

Nobari and Duncan (1972) undertook a detailed investigation of the effects of reservoir filling upon an embankment. Complex movements had been observed in many structures, causing cracks due to differential settlement and hydraulic fracturing. The basic mechanisms are:

(a) compression due to wetting of the foundation, causing settlement which is non-uniform due to the progress of the wetting front,

(b) compression of the wetted portion of a homogeneous or zoned fill, causing settlements and upstream movements and spreading possibly,

(c) downstream movements, due to the water load which is greatest during the later stages of filling. Embankments usually exhibit individualistic variations of the above based upon local conditions and materials.

Two basic mechanisms take place as the reservoir assumes design level. Firstly, before significant seepage patterns are developed, the water load will increase the total stress state in the core. Depending upon the existing construction pore pressures and the core material response (say $\bar{B}$) the effective stress state in the core will usually increase somewhat as well. Secondly, as the steady state seepage pattern forms, there will be some changes in effective stresses in the part affected by seepage. These two mechanisms are not likely to be easily separated or identified in general because of any redistribution of stresses during wetting and because of the masking effect of construction pore pressure dissipation.

Mica reservoir (Lake Kinbasket) started filling in April 1973 and reached a level well below minimum pool before discharge requirements under the Columbia Treaty reduced it

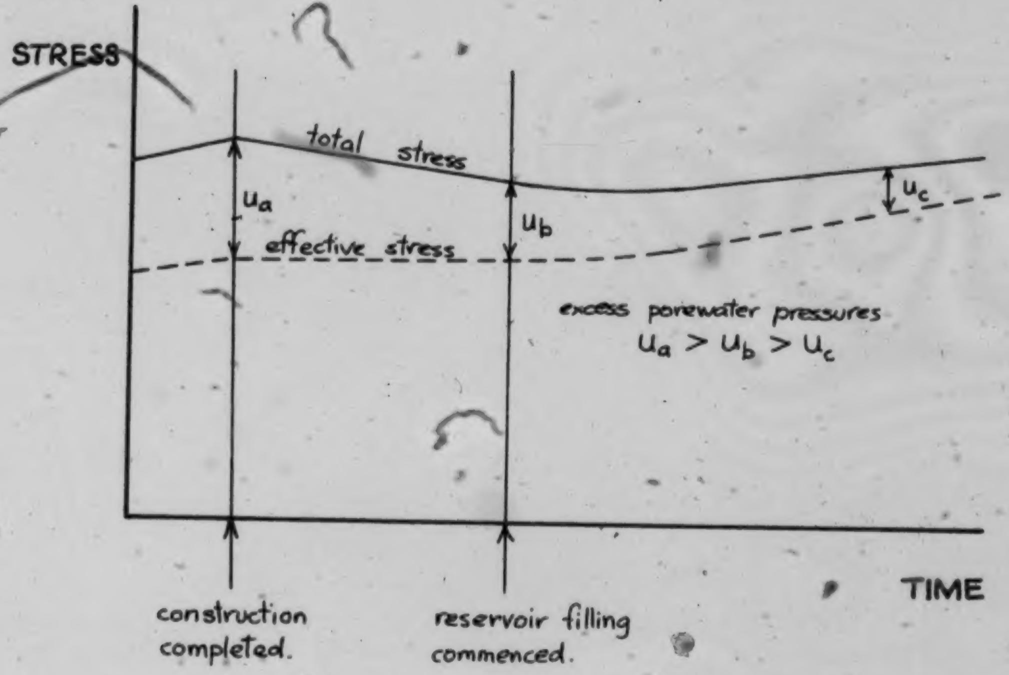
STRESS
total stress
effective stress
u_a
u_b
u_c
excess porewater pressures
$u_a > u_b > u_c$
TIME
construction completed.
reservoir filling commenced.

FIGURE 7.1 DIAGRAMMATIC RESPONSE OF EARTH PRESSURE CELL TO RESERVOIR FILLING

farther. It may fill to within 50 feet of maximum pool by mid-summer of 1974. Preliminary results from some earth-pressure cells monitoring horizontal stresses in the core responded as discussed above and shown diagramatically in Fig. 7.1. (Skermer, personal communication, 1973). It is too early, at the stage of writing, to discuss reservoir filling at Mica Dam in further detail. The subject requires a separate research program due to the complex material behaviour required for finite element modelling. A good outline of the approach required is that given by Nobari and Duncan (1972).

The special care taken in design of the Dam should ensure that reservoir loading will have a beneficial effect on the stress state in the core. Many embankments have been constructed embodying the arch principle in the core but to the writer's knowledge no specific study or quantitative assessment has ever been made of curvature. If for various reasons some cracks do form allowing the percolation of water the core material is "self healing". The initial erosion process washes in materials which will be caught in and eventually seal up the cracks.

7.2 The Likelihood of Hydraulic Fracture

Blight (1973) presented a means of assessing the stress transfer in a narrow core or trench. Some of the parameters required are obtained by "reasonable engineering estimates". One important parameter is the coefficient of lateral earth pressure. Blight makes use of some high values of 1.5 or 2. If K_o is calculated using elastic theory and Poisson's ratio, much lower values may be obtained. Making estimates of the parameters the following results were obtained for the centre of the Mica Dam core at elevation 1860 ft.

K_o	equiv. μ	Stress transfer (%)
2.0	0.66	76.0
1.0	0.5	57.0
0.5	0.33	37.0

Blight's approach was not successfully applied because it cannot agree with the finite element calculations. The estimates may not have been satisfactory, of course.

Blight cited the example of Balderhead Dam (Vaughan et al, 1970) where a hydraulic fracturing phenomenon was observed. His results indicated that piping failure at Balderhead was initiated by a fracture in the core which occurred during construction. This conclusion is not shared by the original reporters nor by Sherard (1973) in his summary of embankment dam cracking, so that Blight's analysis must be viewed circumspectly.

Skermer (1974) reported the results of water inflow tests in the MV 15 gauge at Mica Dam. His evidence supports the reliability of his finite element analysis which yielded very low stresses in the core. Data and water pressures are taken from his work, and minor principal stresses from the present 3D analyses, and shown on Fig. 7.2. Reservoir level was at el. 2218 ft. Water level in the MV gauge could not be raised above et. 2185 ft. despite copious inflow. Undoubtedly hydraulic fracturing occurred at some point around the casing, most likely below el. 1950 ft. where computed minor principal total stresses are less than water pressure. Alteratively the water could have escaped through bedrock as fissures opened up under the high water pressure in the gauge. To the writers' knowledge no information exists as to whether the reservoir level itself caused any hydraulic fracturing. According to the above studies it could have. Perhaps the self-healing properties of the core have closed any such fractures. The questions regarding hydraulic fracturing of the core of Mica Dam are not fully answered at this stage. Under the circumstances, though, leakage due to hydraulic fracturing is not considered likely.

The phenomenon of hydraulic fracturing is excellently covered by Sherard (1973). Deliberate fracturing in boreholes has been utilized in the high pressure Soletanche "sleeve grouting" procedure for grouting alluvium under dams, while Bjerrum and Andersen (1972), Vaughan (1972), and Penman (1972) have discussed the use of induced hydraulic fracturing to estimate the earth pressures in materials (in a similar manner to that illustrated at Mica Dam). Sherard observed

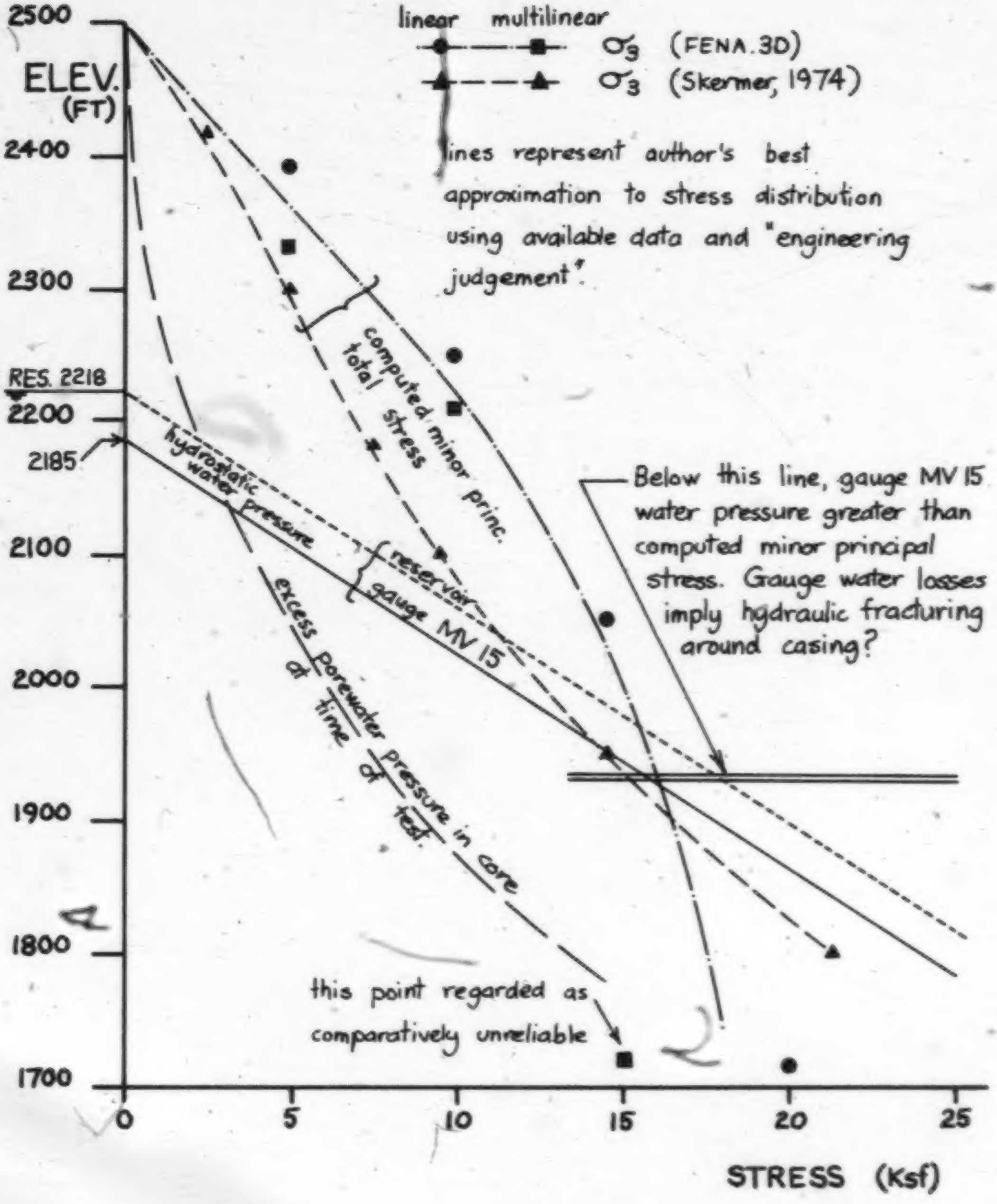

FIGURE 7.2 INFLOW TEST IN GAUGE MV15

that the use of fracturing in boreholes, particularly involving large water inflows, may lead to a dangerous situation where none previously existed. He pointed out the different stress situations of borehole-induced and reservoir-induced hydraulic fracturing.

Sherard, et al (1963) present comprehensive data gathered for earth dams in North America, which shows that, after the non-geotechnical problem of flood-overtopping, piping failures comprise the greatest proportion of unsatisfactory earth dam performance. Evidence from the World Register of Dams supports this conclusion. Sherard, Decker, and Ryker (1972) described the piping failures of several low embankments and concluded that hydraulic fracturing initiated the fissuring which led to the piping. Evidence from finite element analyses supports their argument. It may well be that many more of these "piping failures" are introduced by hydraulic fracturing mechanisms, unrecognized because of the limited knowledge available concerning stress states in embankments. Very few confirmed cases, where hydraulic fracturing has initiated leakage in large dams, have been reported and discussed in a thorough fashion. Finite element analyses may play a fundamental role in furthering the understanding of this process.

7.3 Earthquake Design of Mica Dam

As discussed by Sherard (1967) and Seed (1973), there is a serious lack of information concerning the earthquakeresisting design of high embankments. The limitations of analytical procedures are well illustrated by Seed, who in summarizing states that "Previous performance of earth dams during earthquakes gives no grounds for complacency regarding the ability of well-constructed embankments to withstand the effects of a major earthquake, although, if suitable design measures are taken, they give no ground for concern." Both authors state that the most effective features which can be employed are:

(a) a core of sufficient plasticity and of good self-healing properties,

(b) good filtering properties at the core-shell interfaces,

(c) provision of adequate freeboard, crest width and erosion resistance against overtopping,

(d) minimising of the risk of liquifaction of the upstream shell.

Sherard also gives a summary of the best properties for core material based upon experience and assessment.

Intuitively the most satisfactory stress distribution in an embankment is as uniform across discontinuous zones as possible, and this can be achieved with core and shells of equivalent stiffness. This is invariably impossible to achieve in practice because the upstream shell is required to be as stiff as possible to minimize the risk of liquifaction (Casagrande, 1965). At Mica Dam this was done, and reliance placed upon the self-healing properties of the till core and the good filter-transition properties of the sandy gravel downstream shell to safeguard the core against any crack formation (Skermer, 1974).

The effects which an earthquake would have on Mica Dam, bearing in mind the stress state in the core, are not clearly understood by the writer. With the provisions that have been made in design it may be assumed that the structure would be as safe as it can be regarding reasonable design and extensive experience as against foreseen circumstances. However, the 3D finite element analyses which have been reported in this thesis indicate that a 2D analysis should be quite representative. It would be possible to undertake a 2D dynamic response analysis, given information concerning a possible earthquake, to determine after some fashion stress or strain data which may prove valuable in investigating the possible effects of a shock on the structure. The knowledge of stress and strain data under dynamic loading would be even less substantial than the static case (which is quite limited) so that properties used in such analyses would need to be chosen with care and results treated with caution. Such an exercise may be quite instructive.

CHAPTER VIII

CONCLUSIONS

Four general classes of conclusions can be drawn from the 3D finite element analyses described in this thesis:

(1) Scope, implications, and acceptability of this work within the range of finite element analyses which can be used at Mica Dam.

(2) The role of these analyses in discussing the field behaviour of Mica Dam.

(3) The general validity of such an approach and the sophistication required for analysis.

(4) Specific recommendations for future work.

Scope, Implications and Acceptability

It has been shown that the coarse 3D analysis, using stress-strain data derived mostly from the field, has successfully reproduced aspects of field behaviour. This success of the model suggests that the stresses calculated are representative of conditions in the embankment, and some field evidence may confirm the stress results. As discussed in a previous investigation (Eisenstein, Krishnayya, and Morgenstern, 1972b), there exists useful information which can be utilized for assessing the cracking potential of the core of Mica Dam.

The analyses suggest that despite the markedly 3D aspect of the structure, the constructional deformation and stress behaviour can more economically and just as successfully be obtained from 2D plane strain analyses.

For the materials and loadings considered further evidence has been presented that gross but thoughtful simpliciations in material behaviour can be made and still produce adequate information for design and monitoring purposes.

Reasonable inferences as to the effects of reservoir filling can be made from the present results.

Role of the Analyses in Field Studies

Since most deformational data was obtained from field measurements the agreement of field and analytical settlements is a mark of self-consistency of the data only and indicates that in the core, where laboratory data was used, the finite element results vindicate the simple deformational moduli selected.

If leakage due to hydraulic fracturing is detected, as seems highly unlikely, the analyses should provide very useful information. In advance of any further studies, the results will shed some light on any unexpected behaviour observed during reservoir filling and possibly earthquake loading.

Some information is available concerning stresses and movements in areas where conventional 2D analyses may not be justified, such as zones of potential cracking. It is important to recognize the coarseness of the present 3D coverage in this respect.

General Validity of the Approach

Results of these finite element analyses indicate that even with such gross simplifications in stress-strain representation and mesh geometry successful finite element modelling of Mica Dam occurred. The lack of detail should always be borne in mind, but also bearing in mind the accumulation of finite-element-analysed case histories now available, good average data which can be applied in engineering design and evaluation are now available.

Apparently, the simple linear elastic incremental analyses can provide satisfactory modelling of the structure and for general design purposes any further sophistication is unwarranted except in some severe cases. The data which would be available before construction may cause serious disagreements between predicted and measured behaviour, throwing light on the validity of sampling and testing procedures rather than the calculations themselves. Obviously there is much scope for effective research in developing reliable test methods to represent constructed soil behaviour.

It is worthwhile to bear in mind the comment of Smith and Kay (1971) concerning the apparent success of very crude approximations to true soil behaviour. When applying the finite element method to phenomena where the basic deformational response is pseudo-elastic, we can expect reasonable results using a pseudo-elastic analysis. The effects of consolidation, creep, yield, hysterisis and dynamic loading are a whole world of complexity different, and at the present stage cannot be realistically employed with great reliability in finite element analyses for design purposes. Parametric studies in these fields will continue to be of great use, however, as research develops.

Specific Recommendations for Future Work

The effects of reservoir filling will likely be of a 3D nature owing to valley geometry and core curvature. Such an analysis, taking into account the wetting of the upstream shell, will be valuable and will be a timely contribution to the state-of-the-art not just regarding the mechanics of wetting and filling but particularly in assessment of the effects of curvature and other procedures to optimise wedging action and minimise serious crack formation. Also, due to the great height of the embankment, detailed finite element analyses in 2D using the best means to account for constructional behaviour with test data will contribute significantly to the general confidence in and development of representative test procedures.

Earlier work (e.g. Palmerton and Lefebvre, 1972) has indicated that in certain situations a 3D analysis would be economical enough and useful enough in assessment of 2D work, because of the extra information and more representative modelling. The restrictions on 3D analyses are the data preparation and computer resources which must be involved, and no quick easing of these restrictions can be envisaged.

Finally, it is readily apparent that the finite element is a useful and practical design tool when used sensibly within the limits of its capabilities. Future researchers should bear in mind always the questions of practical application, otherwise vast amounts of computer resources and energy supply are being expended in playing with sophisticated toys. Users of the method in industry must always educate themselves to the developments which occur and be prepared to change their concepts of how things ought to be done. The new analytical power may well have more impact on future sampling and testing, and our basic understanding of geotechnical processes, than is anticipated today.

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APPENDICES

APPENDIX A

COMPARISION OF RESULTS OF FENA.3D WITH LEFEBVRE ET AL (1973)

From the figures herein, it is seen that in general the results of FENA.3D agree well with those of the reference. Stresses plotted from FENA.3D are based on stresses in the elements adjacent to the longitudinal and transverse sections. The writer is unsure as to the plotting procedure used by Lefebvre et al.

Particular attention is drawn to the stress and displacement results in the elements adjacent to the valley wall. Here the finite element formulation should lead to an artificially stiff structure. Lefebvre et al. discuss this situation and indicate that they discarded results from triangular elements at the valley wall for their 2D analyses. The improved accuracy of their 3D element may justify their accepting the results of tetrahedral and prismoidal elements at the walls, but experience with FENA.3D convinced the writer that boundary stresses must be accepted with caution. The shear strengths mobilized in these elements tended to be high and in the analyses of Mica some shear failure occurred. Unfortunately the doubts with which these results can be applied to the real dam prevent us from examining the development of stress transfer at the abutments in greater detail.

Discrepancies in the results can be attributed to a combination of:

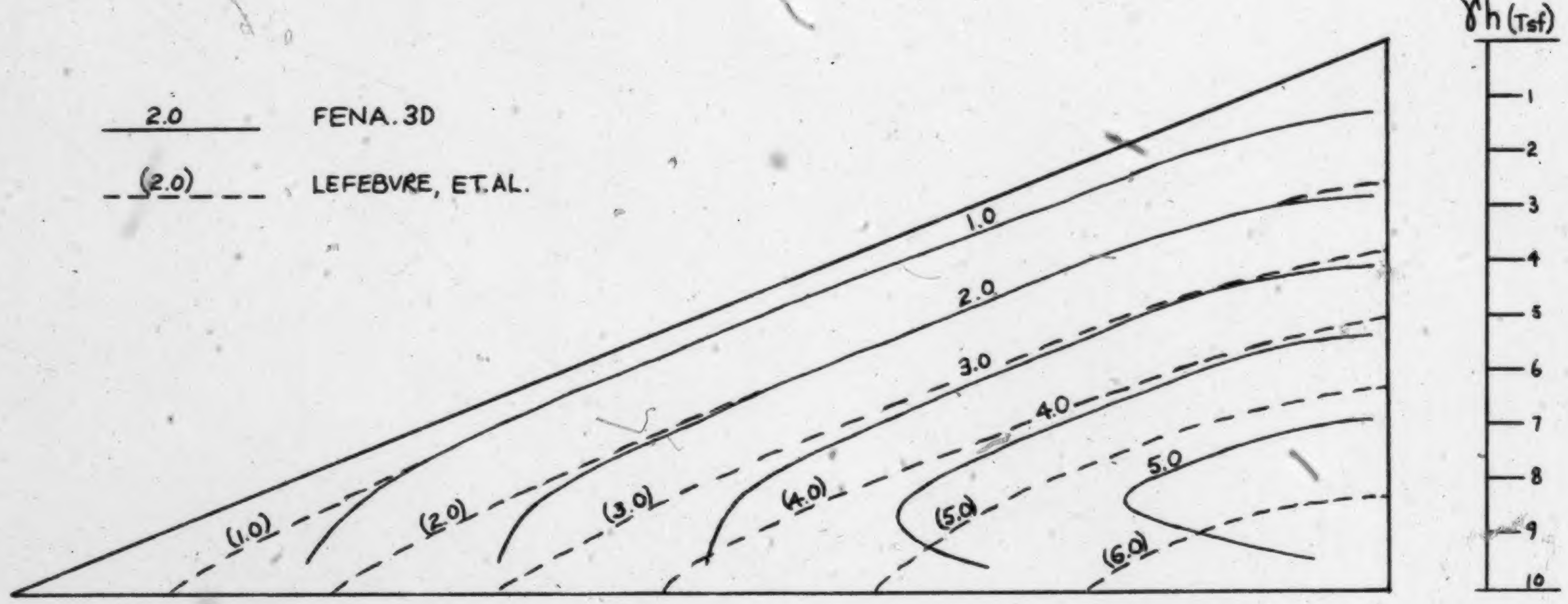
(i) slightly different elements

(ii) logic of the layered analysis

(iii) artificially stiff elements in contact with the valley wall.

(iv) different program algorithms and computer systems.

Results of the analyses indicate arching in the lower central region of the dam of about 28% for FENA.3D and 25% for the other, for the 1:1 valley case. Mica Dam has roughly 2:1 valley walls for which the results of Lefebvre et al indicate about 18% arching. The homogeneous analyses of Mica Dam using FENA.3D yielded about 30% arching due to the 3D valley geometry.



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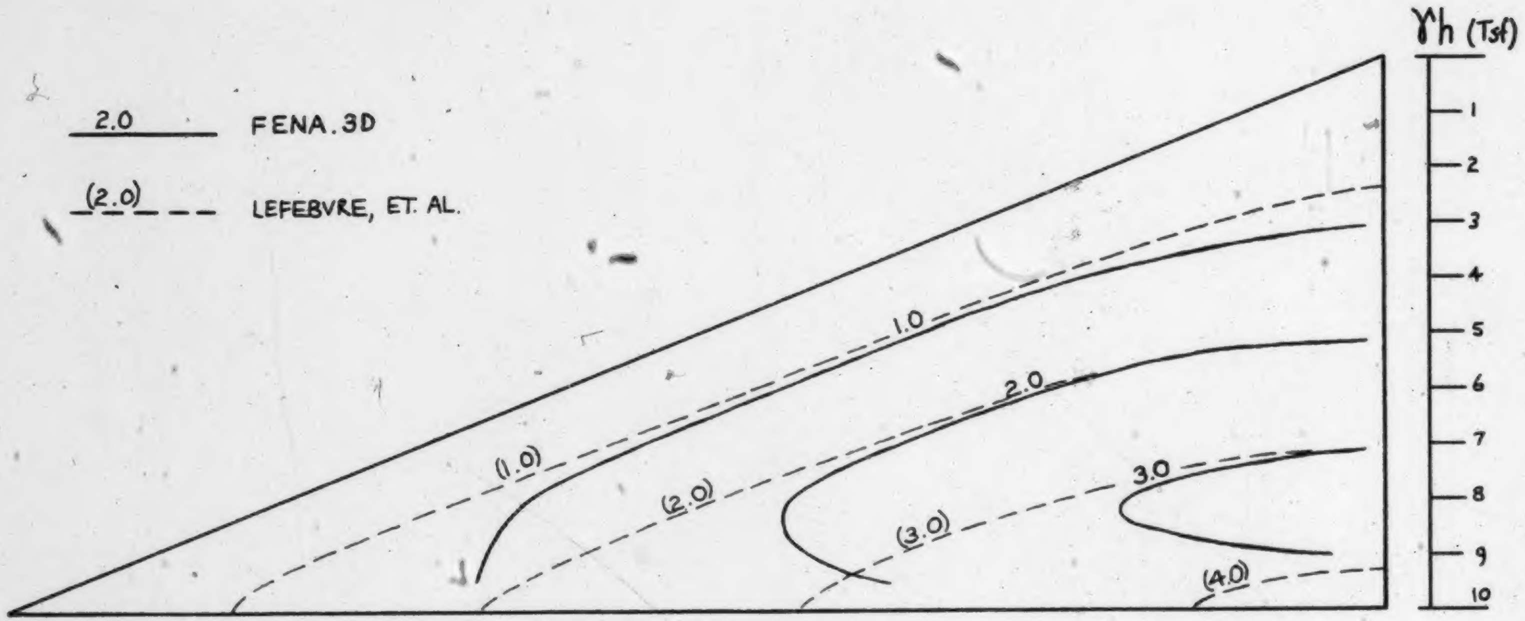

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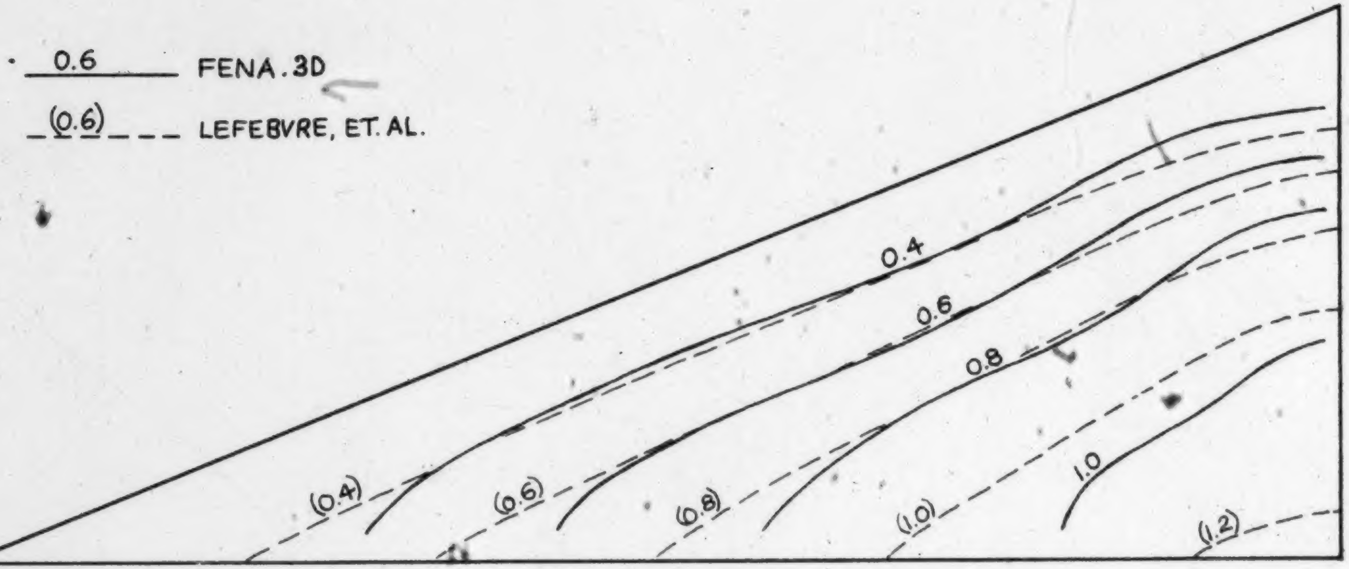

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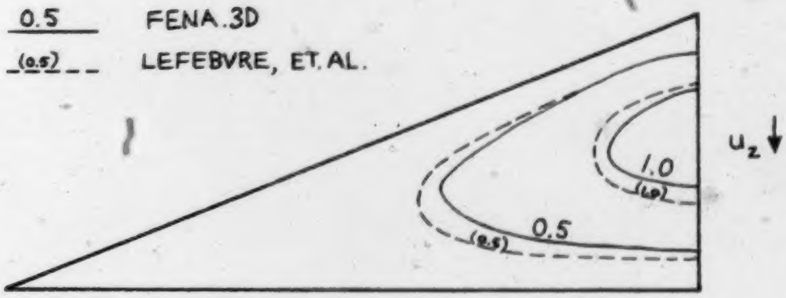
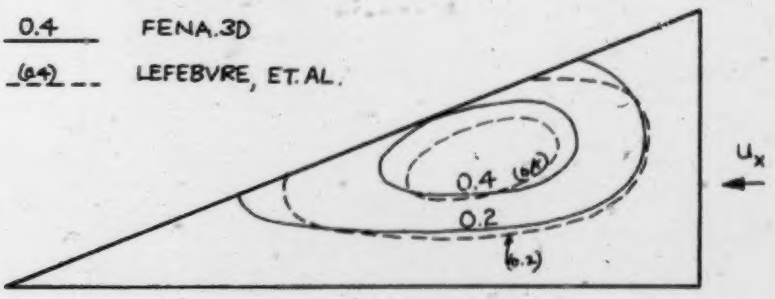
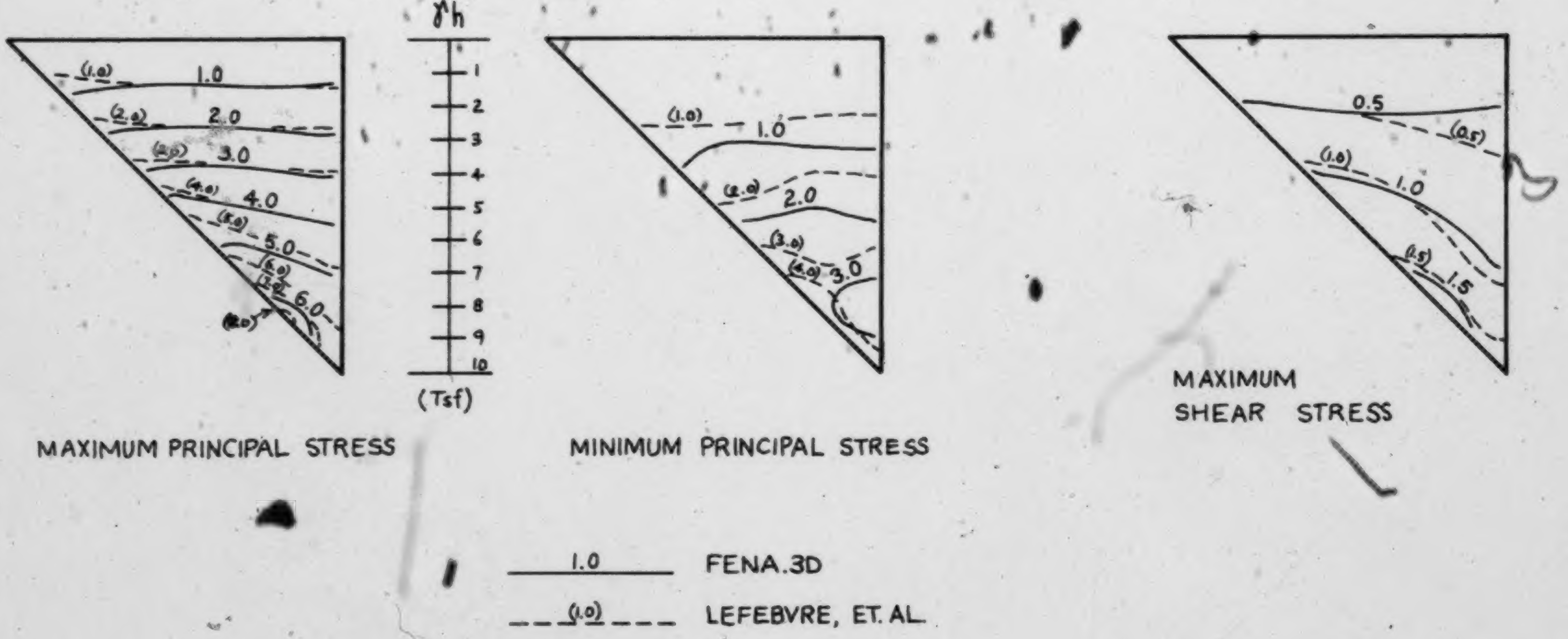


VERTICAL SETTLEMENT



HORIZONTAL MOVEMENT

TRANSVERSE SECTION: HORIZONTAL AND VERTICAL MOVEMENTS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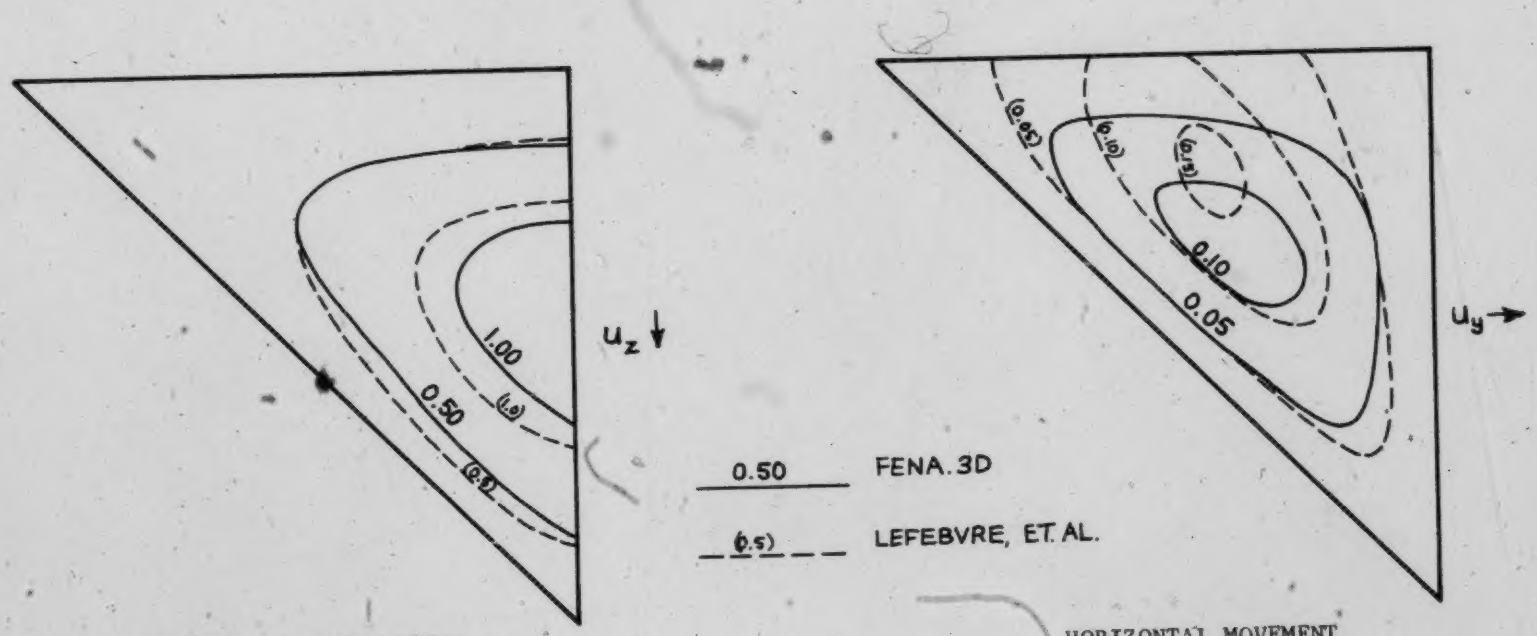

VERTICAL SETTLEMENT

HORIZONTAL MOVEMENT

LONGITUDINAL SECTION: HORIZONTAL AND VERTICAL MOVEMENTS

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APPENDIX B

DETAILS OF THE MICA DAM MESH

Data for the program FENA.3D is prepared as per the users manual (Krishnayya, 1973b). The mesh described in Chapter IV is given in two forms:

(1) the outlines of element boundaries for the different sections are given in Figs. B.1 to B.9. Solid lines are in the vertical plane of the section and dotted lines are not. Nodes are numbered consecutively through the set of planes so as to minimize the bandwidth of equations. Elements are numbered consectively within each layer. These sections can be used in conjunction with,

(2) a full set of nodal data and element data as required for the program FENA.3D. The space on the figures is limited and to avoid confusion only essential numbers are given. This set of data follows the figures B.1 to B.9.

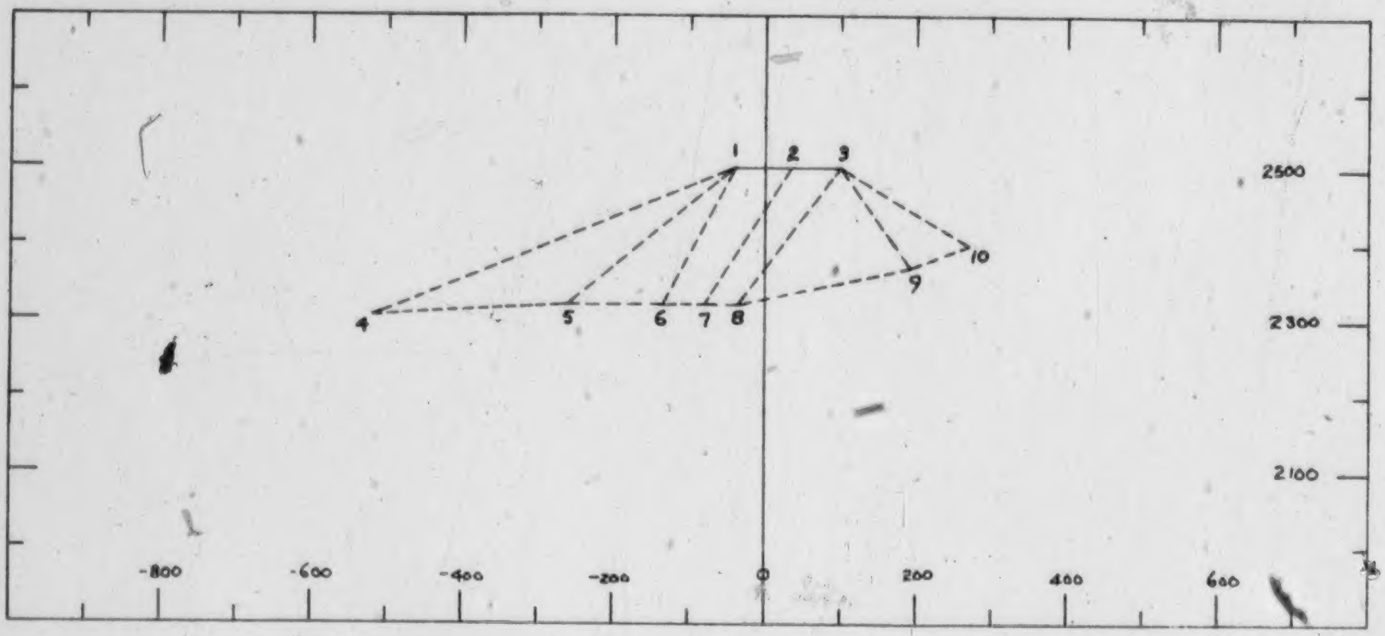
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FIGURE B.1 SECTION 1: SLOPING, RIGHT ABUTMENT

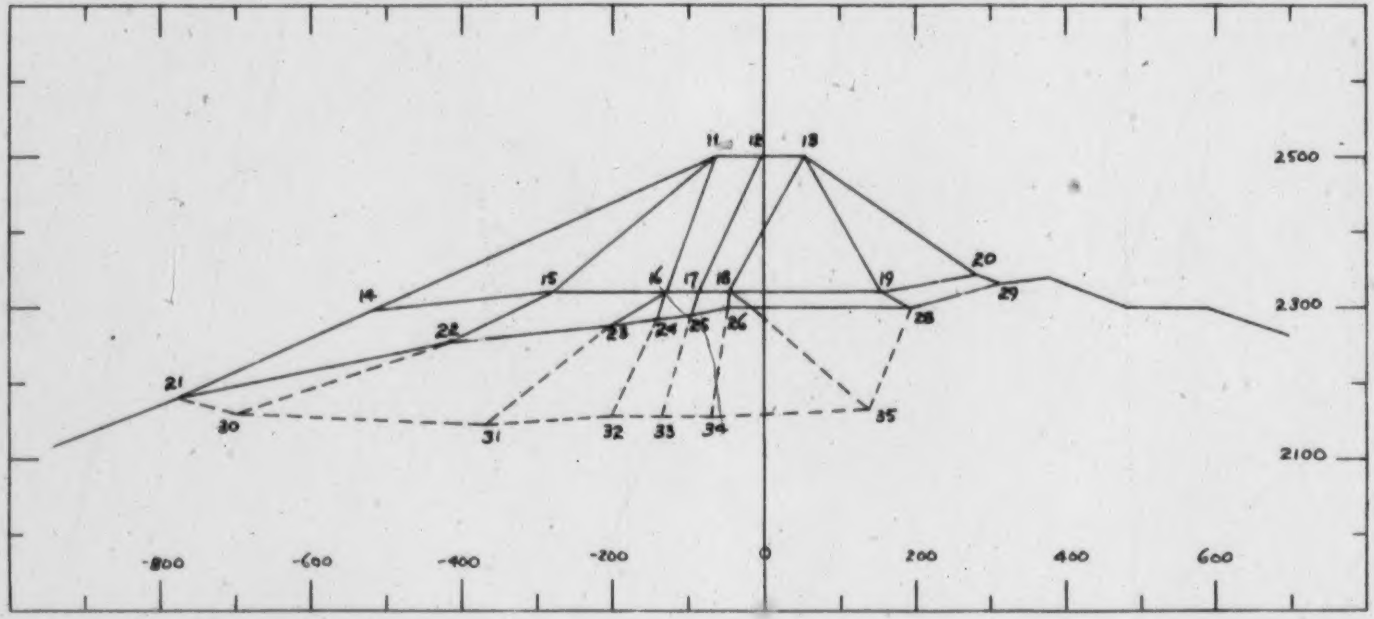
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FIGURE B.2 SECTION 2: STA.15+20

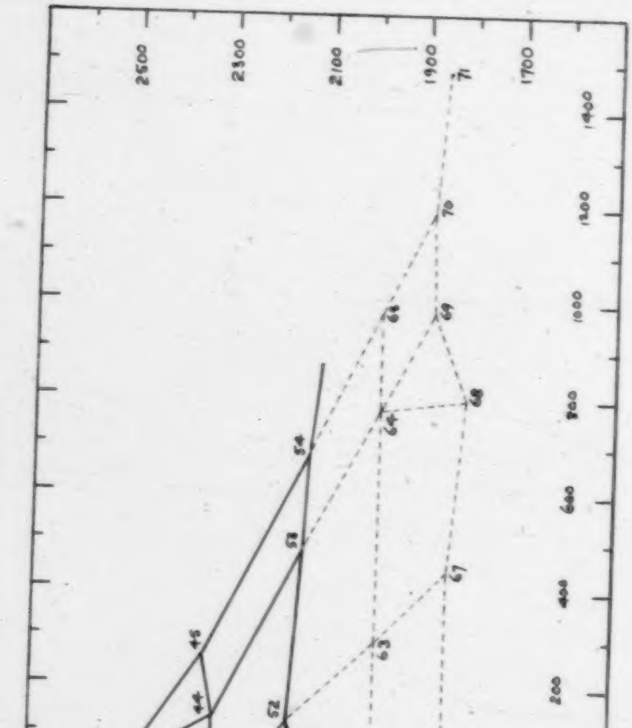
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FIGURE B.3 SECTION 3: STA.16+90

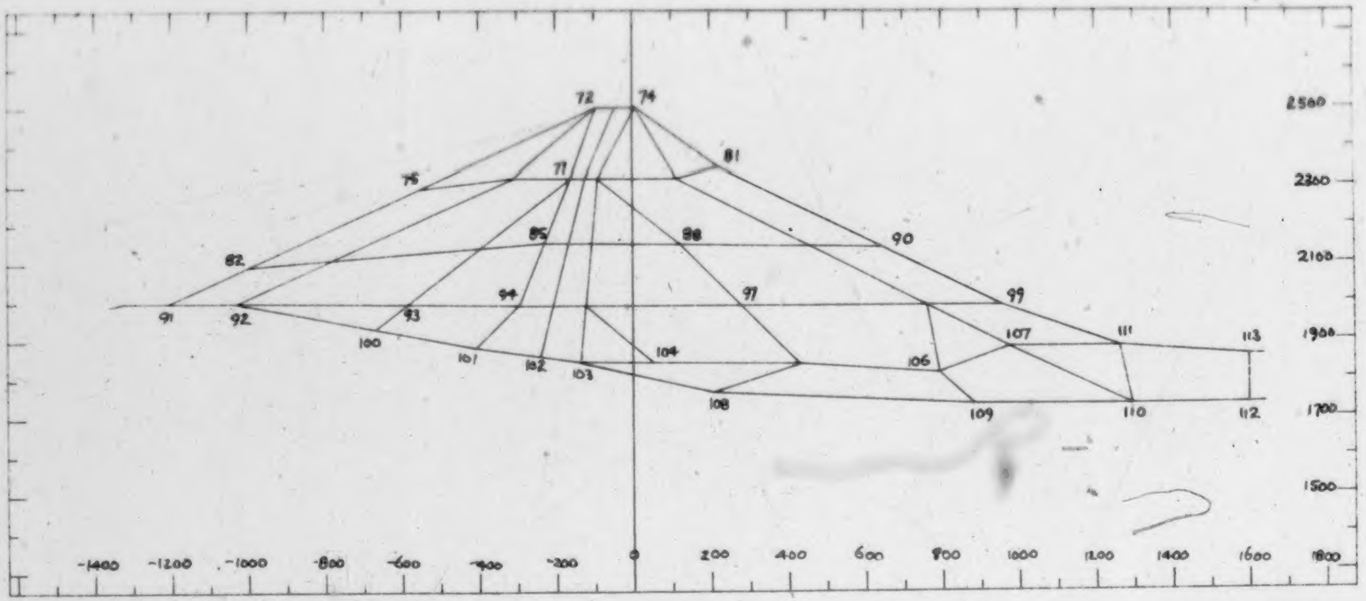
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FIGURE B.4 SECTION 4: STA.21+00

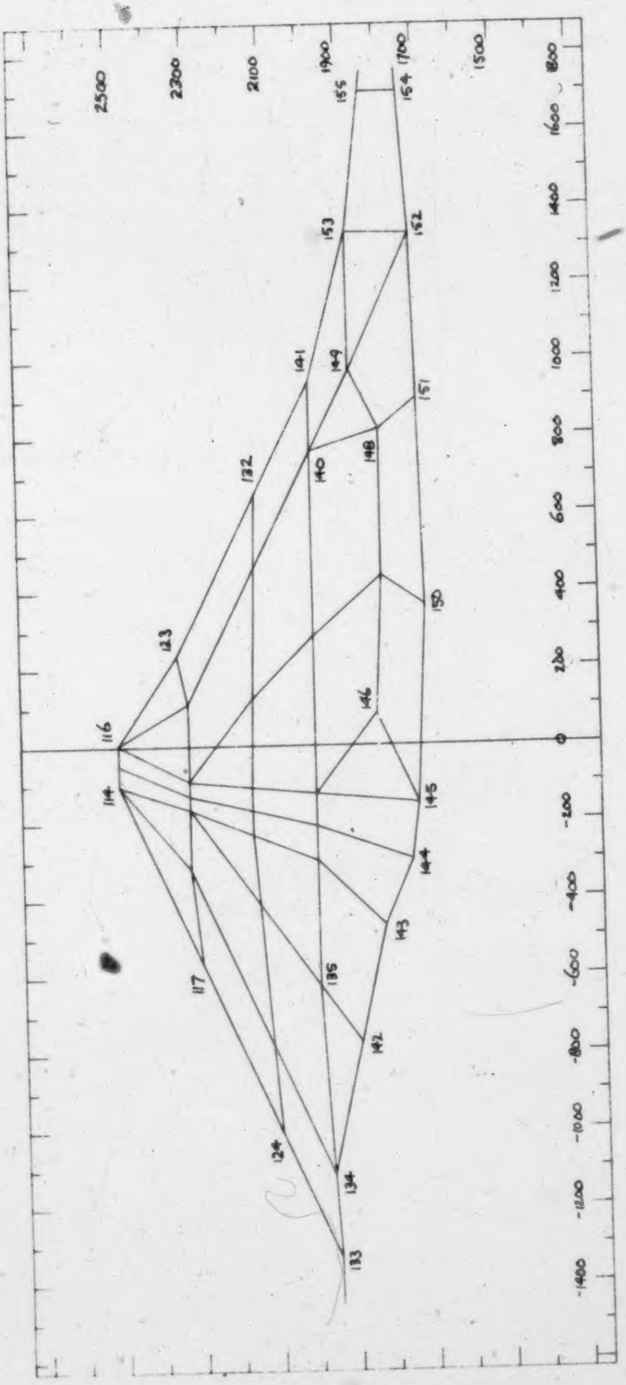
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FIGURE B.5 SECTION 5: STA.22+50

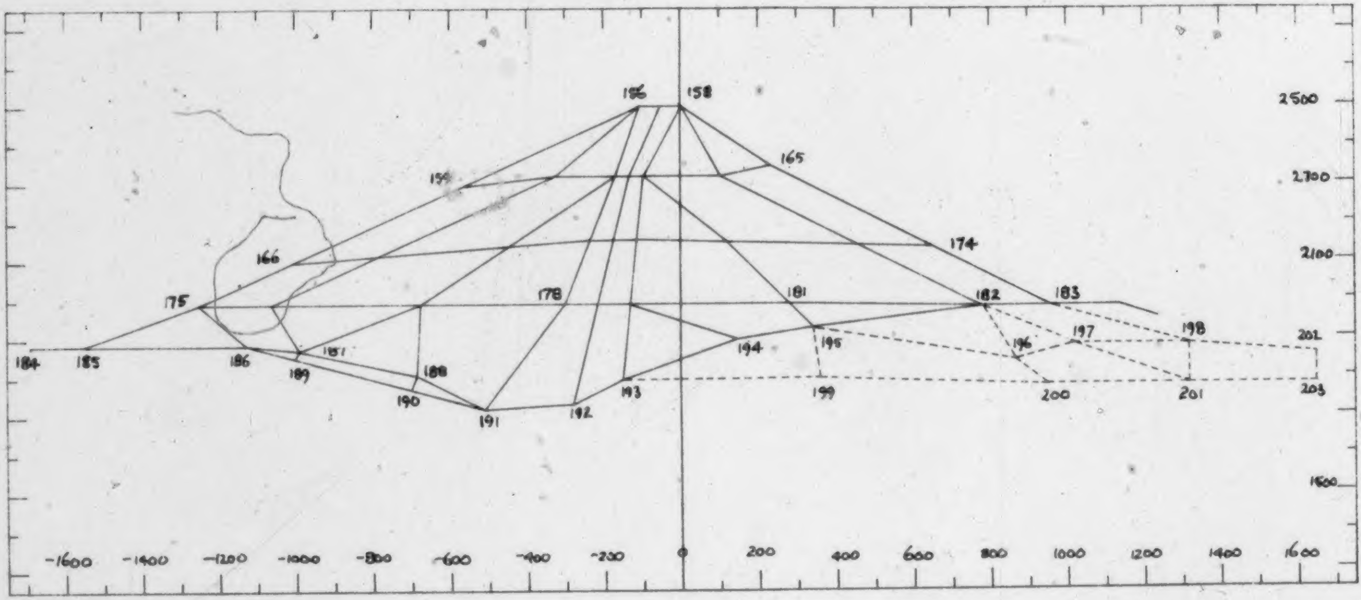
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FIGURE B.6 SECTION 6: STA.25+50

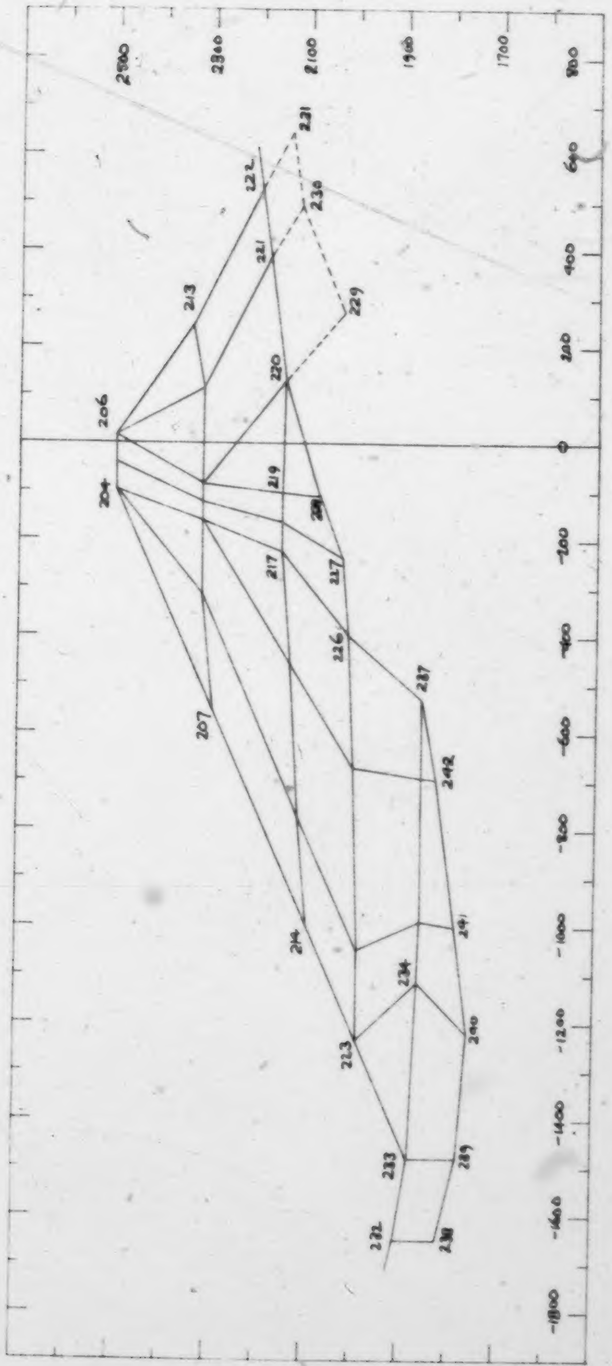
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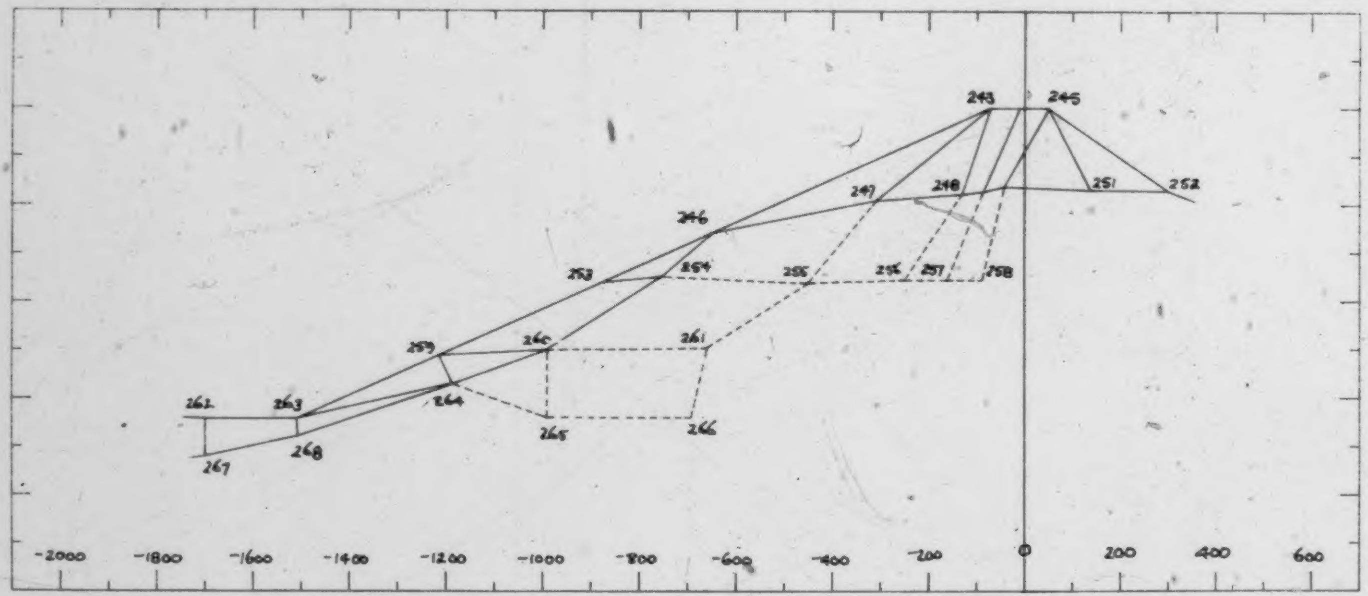
FIGURE B.7 SECTION 7: STA.29+50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256
257
258
259
260
261
262
263
264
265
266
267
268
-2000
-1800
-1600
-1400
-1200
-1000
-800
-600
-400
-200
0
200
400
600

FIGURE B.8 SECTION 8: STA.33+80

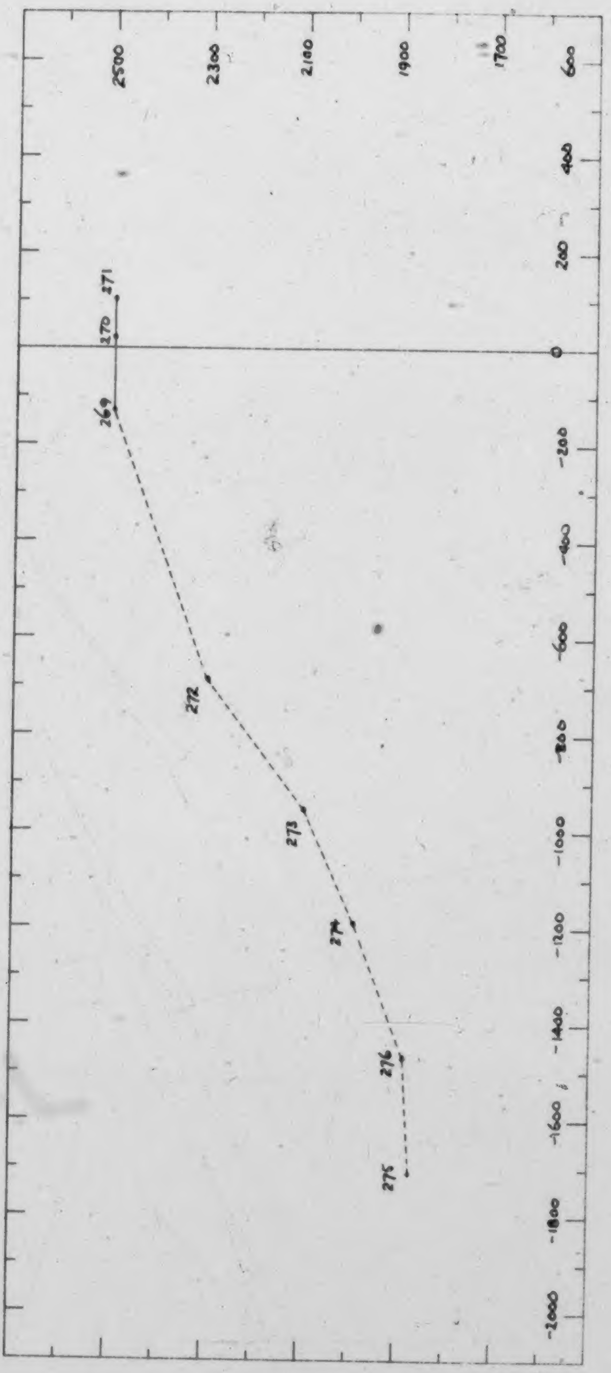

FIGURE B.9 SECTION 9: SLOPING, LEFT ABUTMENT

```
MICA DAM STRESS ANALYSIS - 5 LIFTS
    276    254      7    254      0      0      1
     NO. KODE   X     Y    Z
      1   12   -40  1210  2500
      2   12    35  1210  2500
      3   12   100  1210  2500
      4   12  -520  1400  2305
      5   12  -260  1470  2320
      6   12  -133  1470  2320
      7   12   -78  1480  2320
      8   12   -35  1490  2320
      9   12   193  1480  2370
     10   12   270  1455  2400
     11    0   -65  1520  2500
     12    0    -6  1520  2500
     13    0    54  1520  2500
     14    0  -521  1520  2295
     15    0  -280  1520  2320
     16    0  -128  1520  2320
     17    0   -86  1520  2320
     18    0   -46  1520  2320
     19    0   157  1520  2320
     20    0   284  1520  2345
     21   12  -777  1520  2180
     22   12  -419  1520  2255
     23   12  -205  1520  2276
     24   12  -142  1520  2283
     25   12   -95  1520  2292
     26   12   -47  1520  2300
     27   12   -17  1520  2300
     28   12   196  1520  2300
     29   12   312  1520  2333
     30   12  -700  1610  2160
     31   12  -370  1645  2145
     32   12  -200  1610  2155
     33   12  -135  1620  2155
     34   12   -70  1642  2155
     35   12   140  1635  2166
     36    0   -76  1690  2500
     37    0   -21  1690  2500
     38    0    34  1690  2500
     39    0  -538  1690  2295
     40    0  -295  1690  2320
     41    0  -140  1690  2320
     42    0  -104  1690  2320
     43    0   -66  1690  2320
     44    0   136  1690  2320
     45    0   265  1690  2345
     46   12- 1020  1690  2080
     47   12  -740  1690  2120
     48    0  -378  1690  2144
     49    0  -206  1690  2155
     50    0  -145  1690  2155
     51    0   -83  1690  2155
     52   12   130  1690  2167
     53   12   492  1690  2140
     54   12   690  1690  2130
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  55    12 -440  1690  2097
  56    12 -236  1690  2083
  57    12 -162  1690  2090
  58    12  -88  1690  2096
  59    12 -580  1960  1990
  60    12 -340  1875  1990
  61    12 -195  1830  1990
  62    12 -110  1840  1990
  63    12  295  1845  1990
  64    12  775  1800  1990
  65    12  980  1770  1990
  66    12   60  2060  1845
  67    12  440  1985  1845
  68    12  800  2020  1810
  69    12  980  1925  1880
  70    12 1190  1890  1885
  71    12 1480  1920  1860
  72     0  -96  2100  2500
  73     0  -44  2100  2500
  74     0   10  2100  2500
  75     0 -556  2100  2294
  76     0 -310  2100  2320
  77     0 -162  2100  2320
  78     0 -125  2100  2320
  79     0  -89  2100  2320
  80     0  114  2100  2320
  81     0  240  2100  2345
  82     0-1000  2100  2096
  83     0 -775  2100  2114
  84     0 -396  2100  2142
  85     0 -230  2100  2155
  86     0 -166  2100  2155
  87     0 -105  2100  2155
  88     0  126  2100  2150
  89     0  460  2100  2144
  90     0  645  2100  2140
  91    12-1210  2100  2000
  92    12-1026  2100  2000
  93     0 -587  2100  1994
  94     0 -296  2100  1990
  95     0 -207  2100  1990
  96     0 -122  2100  1990
  97     0  286  2100  1990
  98     0  764  2100  1990
  99     0  944  2100  1990
 100    12 -667  2100  1933
 101    12 -406  2100  1884
 102    12 -240  2100  1860
 103    12 -136  2100  1845
 104     0   56  2100  1843
 105     0  434  2100  1840
 106     0  790  2100  1815
 107     0  974  2100  1885
 108    12  210  2100  1768
 109    12  884  2100  1734
 110    12 1296  2100  1733
 111     0 1265  2100  1885
 112    12 1600  2100  1734
 113     3 1600  2100  1860
 114     0 -104  2250  2500
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115   0  -49 2250 2500
116   0    6 2250 2500
117   0 -568 2250 2293
118   0 -320 2250 2320
119   0 -166 2250 2320
120   0 -128 2250 2320
121   0  -90 2250 2320
122   0  110 2250 2320
123   0  237 2250 2345
124   0-1010 2250 2095
125   0 -784 2250 2114
126   0 -414 2250 2140
127   0 -232 2250 2155
128   0 -170 2250 2155
129   0 -108 2250 2155
130   0  124 2250 2150
131   0  464 2250 2143
132   0  652 2250 2138
133  12-1330 2250 1950
134  12-1113 2250 1964
135   0 -625 2250 1990
136   0 -300 2250 1990
137   0 -211 2250 1990
138   0 -124 2250 1990
139   0  284 2250 1990
140   0  767 2250 1990
141   0  944 2250 1990
142  12 -770 2250 1885
143  12 -468 2250 1818
144  12 -298 2250 1746
145  12 -150 2250 1725
146   0   80 2250 1830
147   0  438 2250 1810
148   0  820 2250 1810
149   0  976 2250 1885
150  12  360 2250 1700
151  12  900 2250 1710
152  12 1330 2250 1720
153   0 1335 2250 1885
154  12 1700 2250 1740
155   3 1700 2250 1860
156   0 -106 2550 2500
157   0  -50 2550 2500
158   0    4 2550 2500
159   0 -562 2550 2296
160   0 -325 2550 2320
161   0 -170 2550 2320
162   0 -134 2550 2320
163   0  -96 2550 2320
164   0  104 2550 2320
165   0  234 2550 2345
166   0-1010 2550 2096
167   0 -786 2550 2114
168   0 -445 2550 2139
169   0 -236 2550 2155
170   0 -174 2550 2155
171   0 -113 2550 2155
172   0  120 2550 2150
173   0  458 2550 2142
174   0  646 2550 2138
```

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175      0-1250  2550  1990
176      0-1063  2550  1990
177      0 -674  2550  1990
178      0 -303  2550  1990
179      0 -215  2550  1990
180      0 -130  2550  1990
181      0  280  2550  1990
182     12  780  2550  1980
183     12  962  2550  1980
184     12-1690  2550  1885
185     [illegible]2-1550  2550  1885
186     [illegible]2-1128  2550  1885
187      0 -990  2550  1872
188      0 -686  2550  1806
189     12-1000  2550  1850
190     12 -700  2550  1769
191     12 -510  2550  1717
192     12 -280  2550  1730
193     12 -150  2550  1795
194     12  145  2550  1900
195     12  340  2550  1930
196     12  868  2410  1845
197     12 1020  2410  18[illegible]
198     12 1320  2380  1885
199     12  360  2420  1800
200     12  950  2340  1780
201     12 1320  2280  1780
202     10 1650  2380  1860
203     12 1650  2300  1780
204      0  -95  2950  2500
205      0  -40  2950  2500
206      0   15  2950  2500
207      0 -550  2950  2296
208      0 -313  2950  2320
209      0 -159  2950  2320
210      0 -122  2950  2320
211      0  -85  2950  2320
212      0  116  2950  2320
213      0  243  2950  2345
214      0-1000  2950  2096
215      0 -778  2950  2113
216      0 -455  2950  2136
217      0 -225  2950  2155
218      0 -164  2950  2155
219      0 -102  2950  2155
220     12  131  2950  2150
221     12  387  2950  2183
222     12  530  2950  2202
223      0-1[illegible]6  2950  1990
224      0-1048  2950  1990
225      0 -670  2950  2004
226     12 -400  2950  2014
227     12 -240  2950  2028
228     12 -110  2950  2077
229     12  270  2690  2010
230     12  490  2760  2120
231     12  646  2780  2140
232      3-1650  2950  1907
233      0-1480  2950  1880
234      0-1114  2950  1864
```

```
235    0 -989 2950 1860
236    0 -692 2950 1860
237   12 -530 2950 1860
238   12-1650 2950 1820
239   12-1480 2950 1780
240   12-1210 2950 1760
241   12-1000 2950 1788
242   12 -695 2950 1833
243    0  -70 3380 2500
244    0  -10 3380 2500
245    0   50 3380 2500
246   12 -646 3380 2243
247   12 -311 3380 2310
248   12 -132 3380 2320
249   12  -86 3380 2327
250   12  -42 3380 2336
251   12  139 3380 2330
252   12  300 3380 2330
253    0 -880 3380 2140
254   12 -753 3380 2154
255   12 -448 3114 2138
256   12 -250 3105 2145
257   12 -162 3084 2145
258   12  -88 3057 2145
259    0-1214 3380 1990
260   12 -992 3380 2000
261   12 -660 3153 2005
262    3-1700 3380 1857
263    0-1510 3380 1857
264   12-1186 3380 1930
265   12 -990 3187 1860
266   12 -690 3018 1860
267   12-1700 3380 1780
268   12-1506 3380 1820
269   12 -130 3810 2500
270   12   22 3810 2500
271   12  100 3810 2500
272   12 -690 3530 2298
273   12 -955 3430 2095
274   12-1190 3460 1990
275   12-1710 3485 1870
276   12-1470 3615 1885
 NO.  KODE   N1     N2     N3     N4     N5     N6     N7     N8   MATNO.
  1     2   103    103    108    108     66    104    105     67     1
  2     2   108    108    109    109     67    105    106     68     1
  3     2   109    109    110    110     68    106    107     69     1
  4     2    69    110    110     69    107    107    111    111     1
  5     2    70    110    110     70     69     69    111    111     1
  6     2   110    110    112    112     70    111    113     71     1
  7     3   103    145    150    108    104    146    147    105     1
  8     3   108    150    151    109    105    147    148    106     1
  9     3   109    151    152    110    106    148    149    107     1
 10     2   107    149    152    110    111    153    153    111     1
 11     3   110    152    154    112    111    153    155    113     1
 12     2   185    185    186    186    133    175    175    133     1
 13     2   134    189    189    134    186    186    187    187     1
 14     2   134    189    190    142    187    187    188    188     1
 15     2   142    190    190    142    188    188    191    191     1
 16     3   145    193    199    150    146    194    195    147     1
 17     3   150    199    200    151    147    195    196    148     1
```

18	3	151	200	201	152	148	196	197	149	1
19	2	149	197	201	152	153	198	198	153	1
20	3	152	201	203	154	153	198	202	155	1
21	2	233	239	238	232	185	185	184	184	1
22	2	185	233	234	186	175	223	223	175	1
23	2	234	240	239	233	186	186	185	185	1
24	2	235	241	240	234	187	189	186	186	1
25	3	187	189	190	188	235	241	242	236	1
26	2	190	242	237	191	188	236	236	188	1
27	3	238	267	268	239	232	262	263	233	1
28	2	239	268	264	240	233	263	264	234	1
29	2	234	240	241	235	264	264	265	265	1
30	2	235	241	242	236	265	265	266	266	1
31	2	242	266	266	242	236	236	237	237	1
32	2	259	274	276	263	264	264	264	264	1
33	2	268	268	264	264	263	276	276	263	1
34	2	262	267	268	263	275	275	276	276	1
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36	2	59	100	101	60	93	93	94	94	7
37	2	101	101	102	102	60	94	95	61	6
38	2	102	102	103	103	61	95	96	62	6
39	2	103	103	104	104	62	96	96	62	5
40	2	66	104	104	66	103	103	62	62	5
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42	3	67	105	106	68	63	97	98	64	7
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44	3	69	107	111	70	64	98	99	65	7
45	2	92	134	142	100	93	135	135	93	7
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47	3	101	143	144	102	94	136	137	95	6
48	3	102	144	145	103	95	137	138	96	6
49	2	103	145	146	104	96	138	138	96	5
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77	2	224	235	236	225	260	260	261	261	7

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79	2	225	236	237	226	261	266	266	261	7
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87	2	58	58	35	35	34	52	52	34	4
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189	3	170	218	219	171	162	210	211	163	3
190	2	171	219	220	172	163	211	211	163	4
191	3	172	220	221	173	163	211	212	164	2
192	2	172	220	230	173	221	221	221	221	2
193	3	173	221	222	174	164	212	213	165	2
194	2	173	230	231	174	221	221	222	222	2
195	2	214	253	254	215	207	207	208	208	2
196	2	207	253	254	208	246	246	246	246	2
197	2	215	254	255	216	208	208	208	208	2

198	2	255	254	246	247	208	208	208	208	2
199	2	247	208	209	248	255	216	216	255	2
200	2	216	255	256	217	209	248	248	209	2
201	3	217	256	257	218	209	248	249	210	3
202	3	218	257	258	219	210	249	250	211	3
203	2	211	250	258	219	220	220	220	220	4
204	2	220	250	251	221	211	211	212	212	2
205	2	221	251	252	222	212	212	213	213	2
206	2	254	273	272	246	253	253	253	253	2
207	2	4	14	15	5	1	11	11	1	2
208	2	5	15	16	6	1	11	11	1	2
209	3	6	16	17	7	1	11	12	2	3
210	3	7	17	18	8	2	12	13	3	3
211	2	8	18	19	9	3	13	13	3	2
212	2	9	19	20	10	3	13	13	3	2
213	2	14	39	40	15	11	36	36	11	2
214	2	15	40	41	16	11	36	36	11	2
215	3	16	41	42	17	11	36	37	12	3
216	3	17	42	43	18	12	37	38	13	3
217	2	18	43	44	19	13	38	38	13	2
218	2	19	44	45	20	13	38	38	13	2
219	2	39	75	76	40	36	72	72	36	2
220	2	40	76	77	41	36	72	72	36	2
221	3	41	77	78	42	36	72	73	37	3
222	3	42	78	79	43	37	73	74	38	3
223	2	43	79	80	44	38	74	74	38	2
224	2	44	80	81	45	38	74	74	38	2
225	2	75	117	118	76	72	114	114	72	2
226	2	76	118	119	77	72	114	114	72	2
227	3	77	119	120	78	72	114	115	73	3
228	3	78	120	121	79	73	115	116	74	3
229	2	79	121	122	80	74	116	116	74	2
230	2	80	122	123	81	74	116	116	74	2
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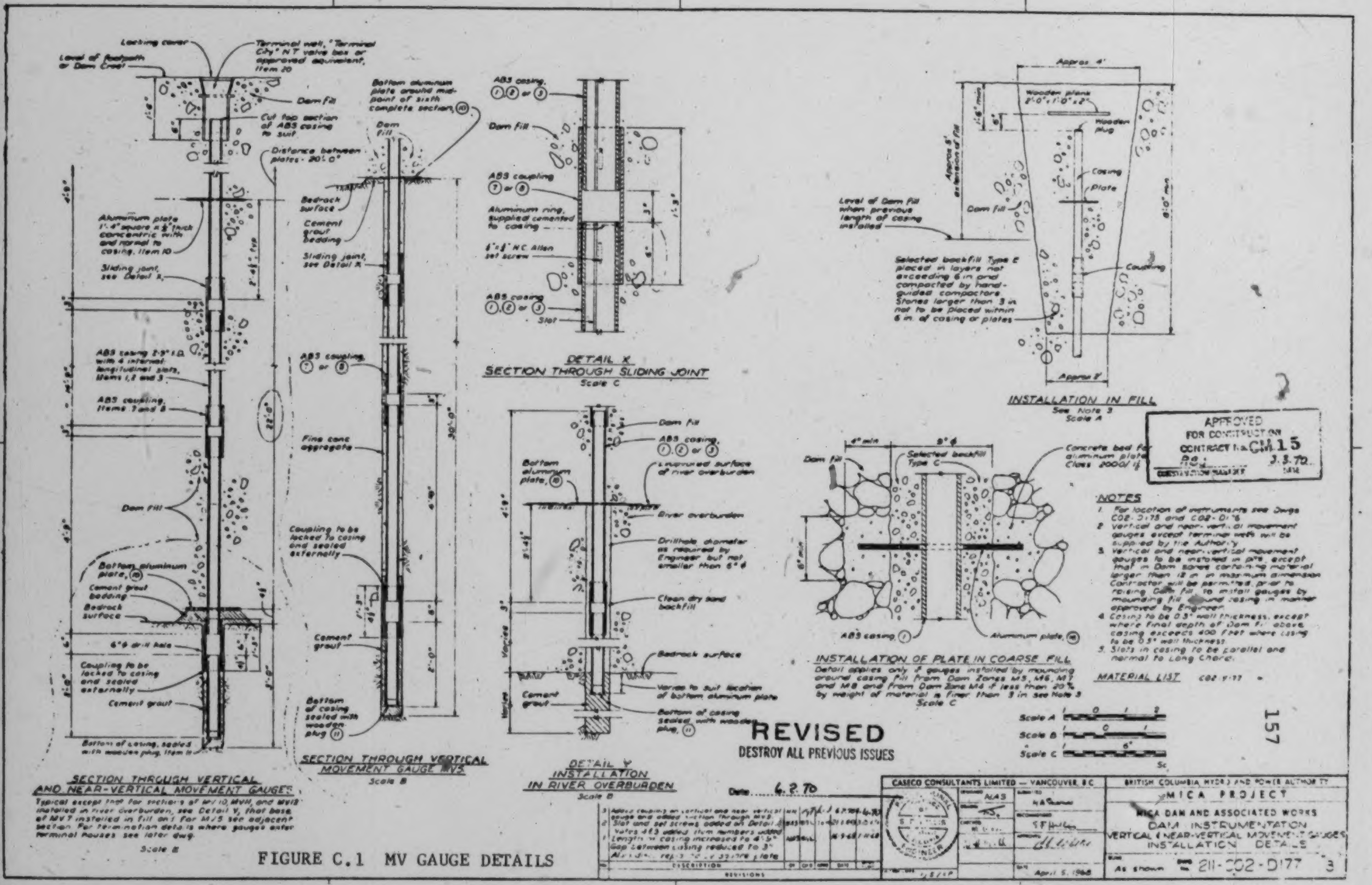
232	2	118	160	161	119	114	156	156	114	2
233	3	119	161	162	120	114	156	157	115	3
234	3	120	162	163	121	115	157	158	116	3
235	2	121	163	164	122	116	158	158	116	2
236	2	122	164	165	123	116	158	158	116	2
237	2	159	207	208	160	156	204	204	156	2
238	2	160	208	209	161	156	204	204	156	2
239	3	161	209	210	162	156	204	205	157	3
240	3	162	210	211	163	157	205	206	158	3
241	2	163	211	212	164	158	206	206	158	2
242	2	164	212	213	165	158	206	206	158	2
243	2	207	246	247	208	204	243	243	204	2
244	2	208	247	248	209	204	243	243	204	2
245	3	209	248	249	210	204	243	244	205	3
246	3	210	249	250	211	205	244	245	206	3
247	2	211	250	251	212	206	245	245	206	2
248	2	212	251	252	213	206	245	245	206	2
249	2	272	246	243	269	247	247	247	247	2
250	2	247	247	248	248	243	269	269	243	2
251	2	243	248	249	244	269	269	270	270	3
252	2	244	249	250	245	270	270	271	271	3
253	2	250	250	251	251	245	270	270	245	2
254	2	251	251	252	252	245	270	270	245	2

APPENDIX C

DETAILS OF MV GAUGES

On succeeding pages are some instrument details supplied by CASECO. The first shows the installation and layout of the telescoping-sectioned MV gauges themselves (Fig. C.1). The second shows the layout of the latch-cone device used in conjunction with a survey tape to measure casing profiles for settlement and incremental strain data. A similar device was used with a hauling cable in horizontal (MH) movement gauges, which were themselves similar in most respects to the MV gauges.

Figure C.2 is a photograph of the latch-cone device. In the closed position, at right it latches under each section of casing. When it reaches the bottom of the gauge, it is dropped to jerk out the springs, as shown on left, that enable it to be drawn up out of the gauge casing.


Locking cover
Level of footpath or Dam Crest
Terminal well, "Terminal City" NT valve box or approved equivalent, Item 20
Dam fill
Cut top section of ABS casing to suit
Distance between plates - 20'-0"
Aluminum plate 1'-4" square x 3/8 inch concentric with and normal to casing, Item 10
Sliding joint, see Detail X
ABS casing 2.9" I.D. with 4 internal longitudinal slots, Items 1, 2 and 3
ABS coupling, Items 7 and 8
Dam fill
Bottom aluminum plate, ⑩
Cement grout bedding
Bedrock surface
6"φ drill hole
Coupling to be locked to casing and sealed externally
Cement grout
Bottom of casing, sealed with wooden plug, Item 11
SECTION THROUGH VERTICAL AND NEAR-VERTICAL MOVEMENT GAUGES
Typical except that for sections of MV10, MV11, and MV12 installed in river overburden, see Detail Y, that base of MV7 installed in fill on 1 for MV5 see adjacent section. For termination details where gauges enter terminal houses see later dwg.
Scale B
Bottom aluminum plate around mid-point of sixth complete section ⑩
Dam fill
Bedrock surface
Cement grout bedding
Sliding joint, see Detail X
ABS coupling ⑦ or ⑧
Fine conc aggregate
Coupling to be locked to casing and sealed externally
Cement grout
Bottom of casing sealed with wooden plug ⑪
SECTION THROUGH VERTICAL MOVEMENT GAUGE MV5
Scale B
ABS casing ①, ② or ③
Dam fill
ABS coupling ⑦ or ⑧
Aluminum ring, supplied cemented to casing
1/4"x1/4" NC Allen set screw
ABS casing ①, ② or ③
Slot
DETAIL X
SECTION THROUGH SLIDING JOINT
Scale C
Dam fill
ABS casing ①, ② or ③
Unimproved surface of river overburden
Bottom aluminum plate ⑩
River overburden
Drillhole diameter as required by Engineer but not smaller than 6"φ
Clean dry sand backfill
Bedrock surface
Varies to suit location of bottom aluminum plate
Cement grout
Bottom of casing sealed with wooden plug ⑪
DETAIL Y
INSTALLATION IN RIVER OVERBURDEN
Scale B
Approx 4'
Wooden plank 2'-0"x1'-0"x2"
Wooden plug
Casing
Plate
Coupling
Level of Dam Fill when previous length of casing installed
Approx 5' extension of fill
Dam fill
Selected backfill Type E placed in layers not exceeding 6 in and compacted by hand-guided compactors. Stones larger than 3 in not to be placed within 6 in. of casing or plates
Approx 8'
INSTALLATION IN FILL
See Note 3
Scale A
Dam fill
Selected backfill Type C
Concrete bed for aluminum plate Class 2000/1½
ABS casing ①
Aluminum plate ⑩
INSTALLATION OF PLATE IN COARSE FILL
Detail applies only if gauges installed by mounding around casing fill from Dam Zones M5, M6, M7 and M8 and from Dam Zone M4 if less than 20% by weight of material is finer than 3 in (see Note 3)
Scale C
Scale A
Scale B
Scale C

APPROVED
FOR CONSTRUCTION
CONTRACT No. CM15
3.3.70.

NOTES
1. For location of instruments see Dwgs C02-D175 and C02-D176
2. Vertical and near-vertical movement gauges except terminal wells will be supplied by the Authority
3. Vertical and near-vertical movement gauges to be installed in pits, except that in Dam zones containing material larger than 12 in in maximum dimension Contractor will be permitted, prior to raising Dam fill, to install gauges by mounding fill around casing in manner approved by Engineer.
4. Casing to be 0.3" wall thickness, except where final depth of Dam fill above casing exceeds 400 feet where casing to be 0.5" wall thickness
5. Slots in casing to be parallel and normal to Long Chord.

MATERIAL LIST C02-Y-77

REVISED
DESTROY ALL PREVIOUS ISSUES

Date 6.2.70

3	Added coupling on vertical and near vertical gauge and added section through MV5			6.2.70
2	Slot and set screws added on Detail X			
1	Notes 4&3 added, item numbers added. Length of casing increased to 4'-5". Gap between casing reduced to 3". Aluminum rep. ... square plate			
No.	DESCRIPTION			DATE
	REVISIONS			

CASECO CONSULTANTS LIMITED — VANCOUVER, B.C.
NAS
S F Hill
April 5, 1968

BRITISH COLUMBIA HYDRO AND POWER AUTHORITY
MICA PROJECT
MICA DAM AND ASSOCIATED WORKS
DAM INSTRUMENTATION
VERTICAL & NEAR-VERTICAL MOVEMENT GAUGES
INSTALLATION DETAILS
As shown 211-C02-D177 3

FIGURE C.1 MV GAUGE DETAILS

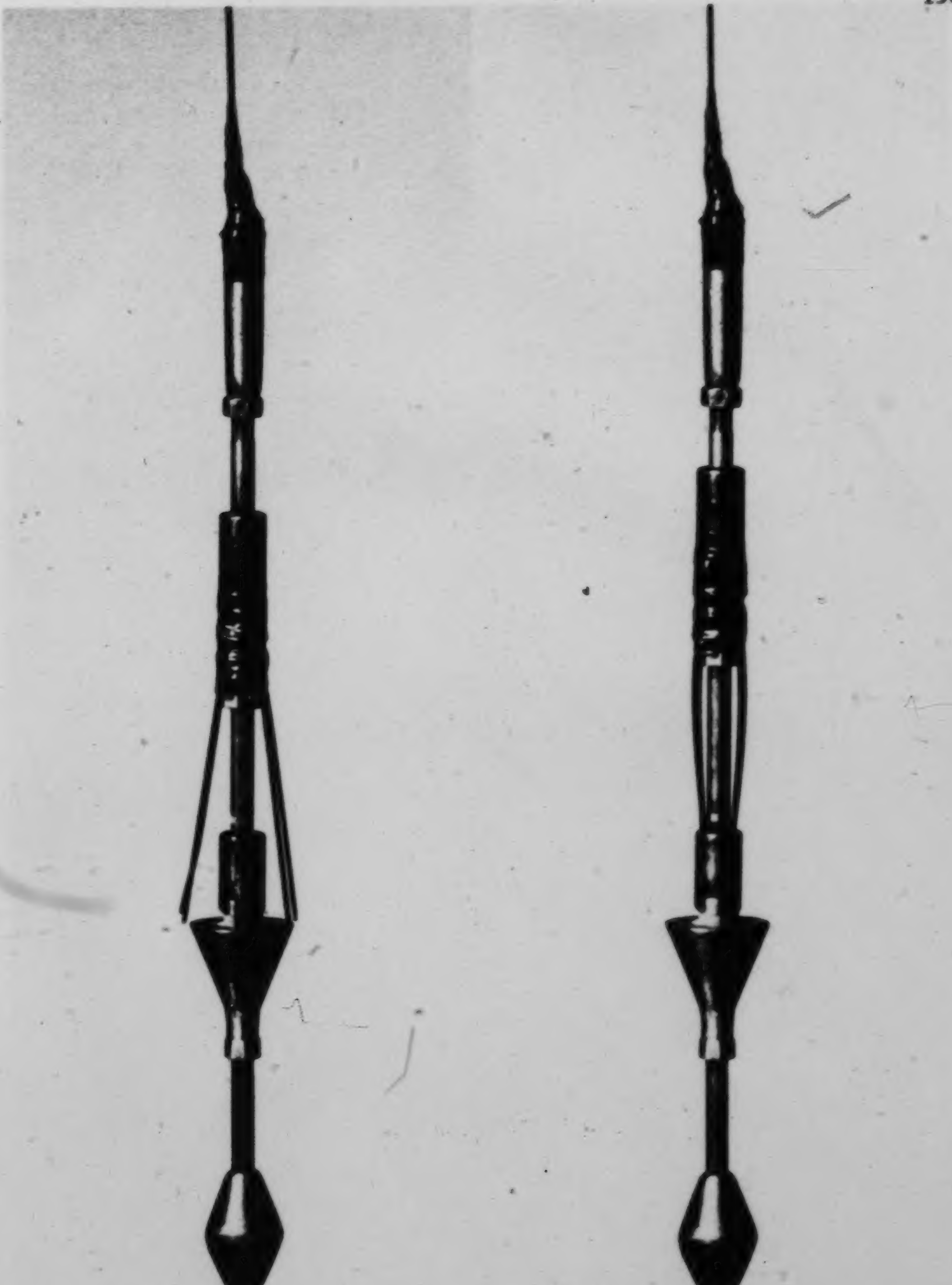


FIGURE C.2 LATCH CONE DEVICE

APPENDIX D

PLOTS OF STRESS DISTRIBUTIONS FROM FENA.3D

Herein are a series of plots of stresses as described in Section 6.3.

Transverse Section, Station 22+50

Plots show major, intermediate, and minor principal stresses and maximum shear stress for the following cases:

(a) 3D linear analysis, 7 materials, 1 lift, bedrock fixed.

(b) 2D linear analysis, plane strain, 7 materials, 5 lifts, bedrock fixed.

(c) 3D linear analysis, 7 materials, 5 lifts, bedrock settlements included.

(d) 3D multilinear analysis, 7 materials, 5 lifts, bedrock settlements included.

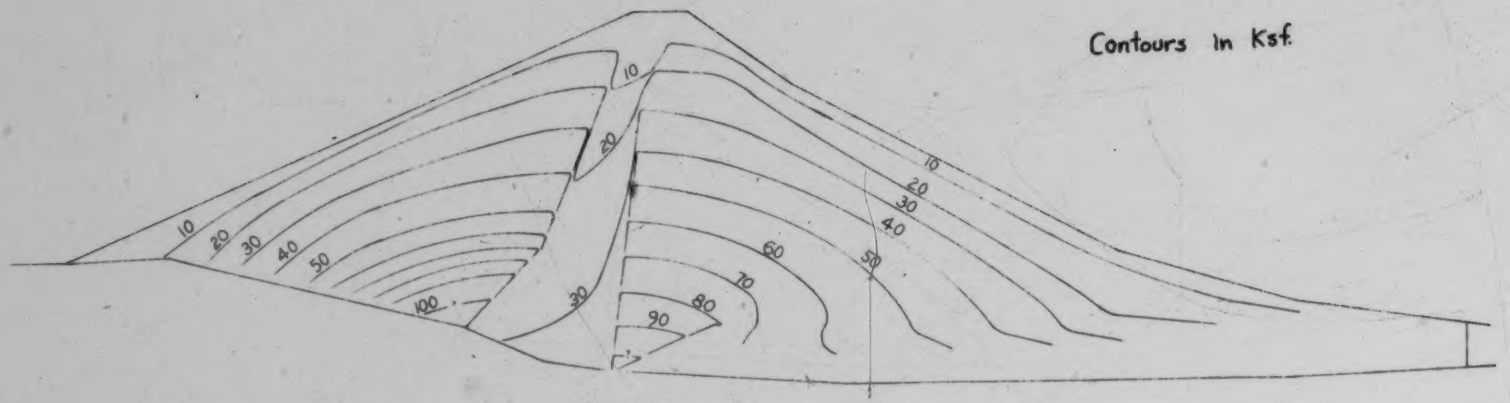
As mentioned previously, the stresses shown are those computed at the element centroids between Stations 22+50 and 25+50.

Longitudinal Core Section

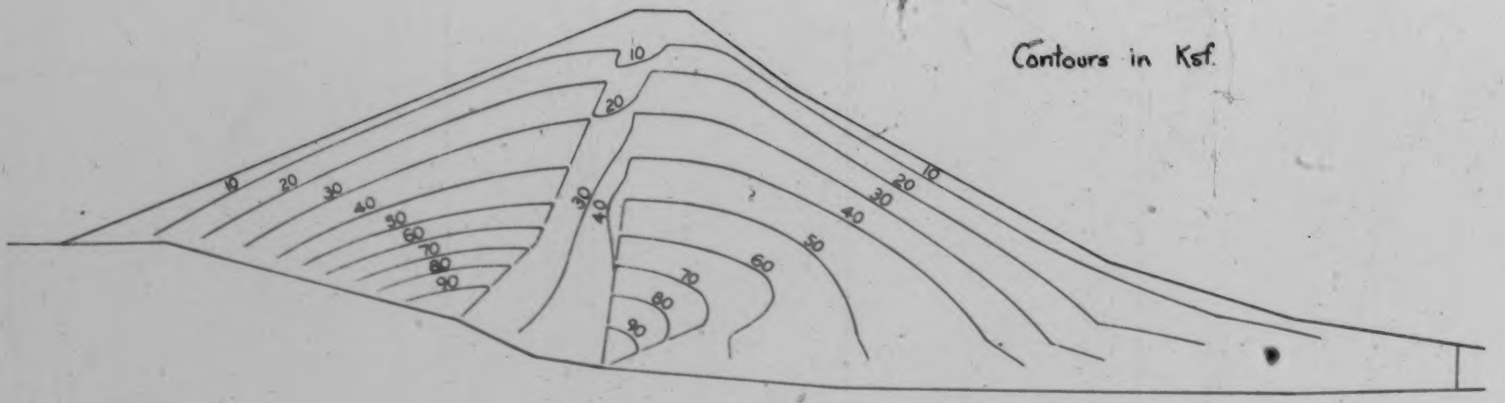
Plots show major, intermediate, and minor principal stresses and maximum shear stress for cases (a), (c), and (d) detailed above. As mentioned previously, the stresses shown are the mid-core average of values computed for the upstream and downstream core elements at a certain section.

The reader is asked to interpret these results in a suitable fashion. Clearly values for the incremental analyses do not differ markedly but the 1 step "gravity switch-on" stresses are not so agreeable. In general, major principal stress tends to align with vertical stress, intermediate principal stress with cross-valley (Y direction) stress and minor principal stress with transverse (X direction) stress respectively, although the trajectories curve in an appropriate manner near boundaries (e.g. Covarrubias, 1969).

The writer concludes that the stresses from the linear analysis including bedrock settlements are the most representative since the best agreement with field deformations was achieved in this computation. The multilinear analysis has too soft a response in the 1969 core material (below el. 1990 ft.).



TRANSVERSE SECTION: MAJOR PRINCIPAL STRESS; 3D, MULTILINEAR


Contours in Ksf.
10
20
30
40
50
60
70
80
90
10
20
30
40
10
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30
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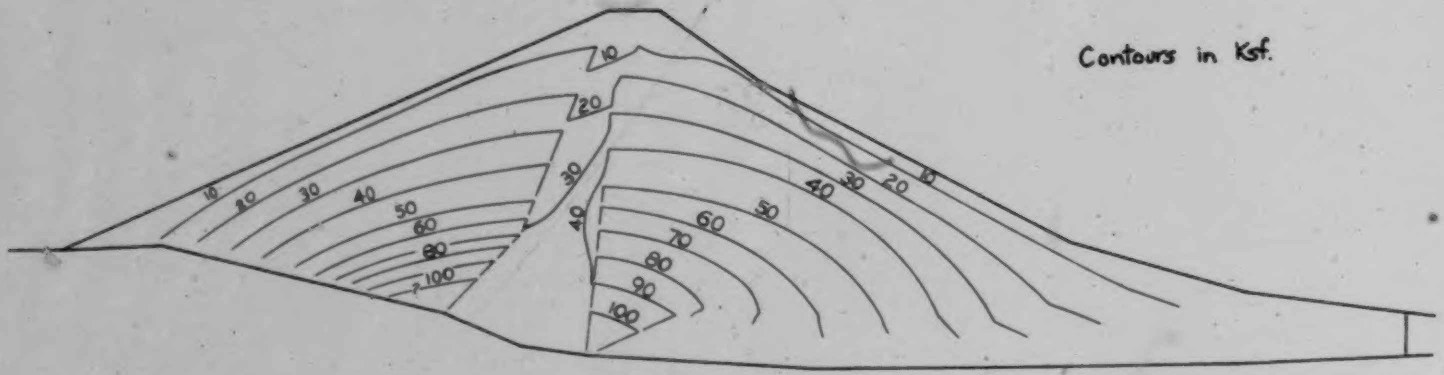
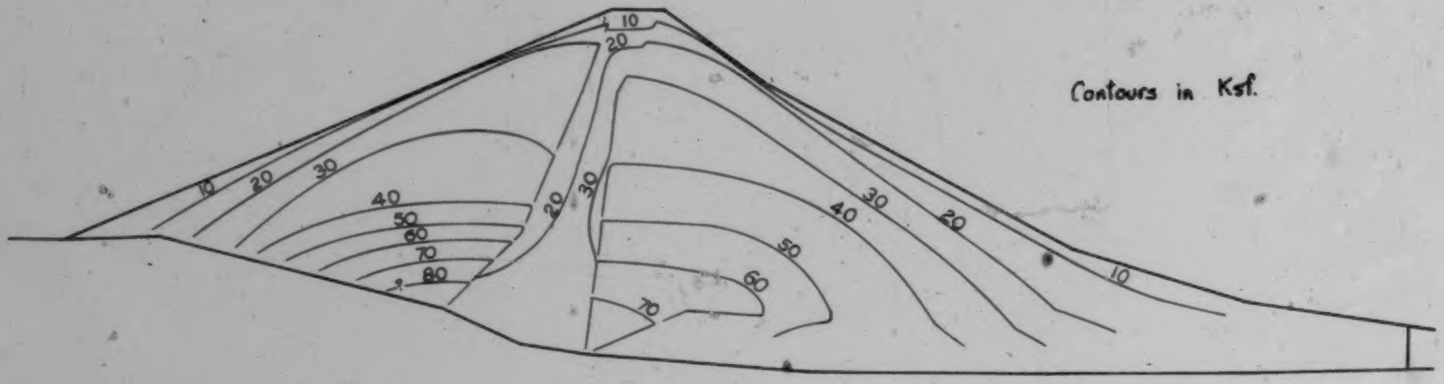
50
60
70
80
90

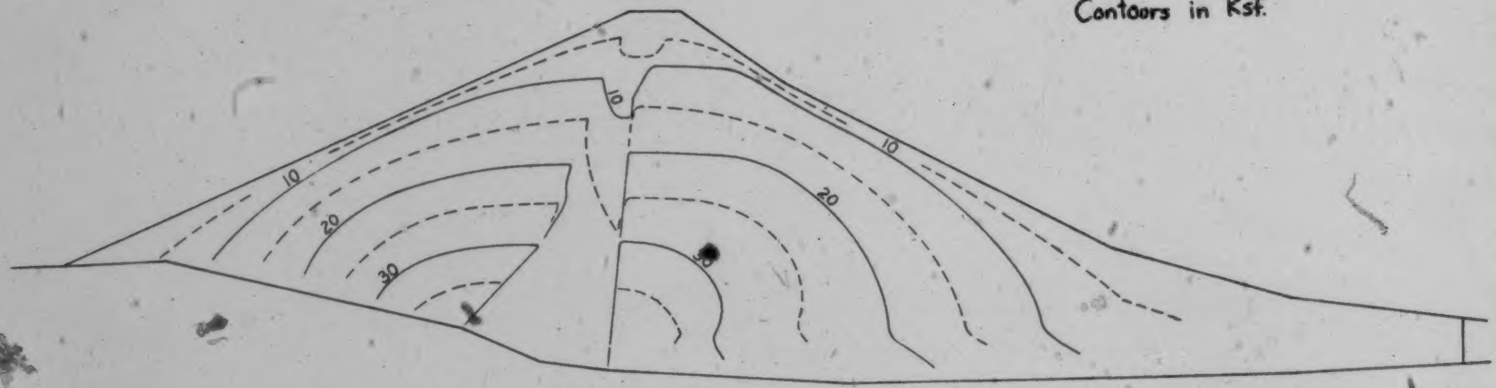
TRANSVERSE SECTION: MAJOR PRINCIPAL STRESS; 3D, LINEAR+BEDROCK



TRANSVERSE SECTION: MAJOR PRINCIPAL STRESS; 2D, PLANE STRAIN



TRANSVERSE SECTION: MAJOR PRINCIPAL STRESS; 3D, LINEAR, 1 LIFT


10
20
30
10
10
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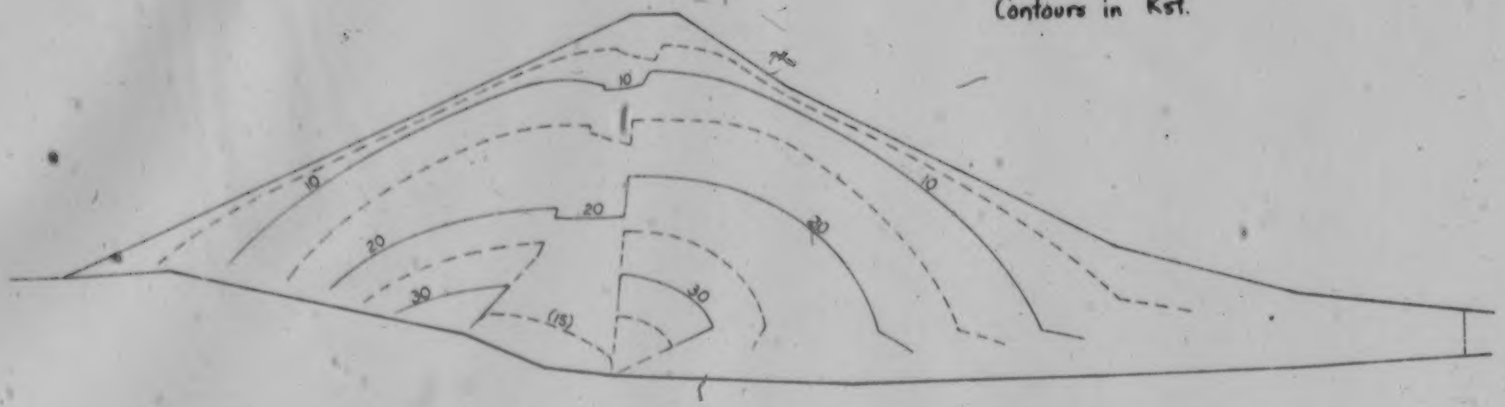
30

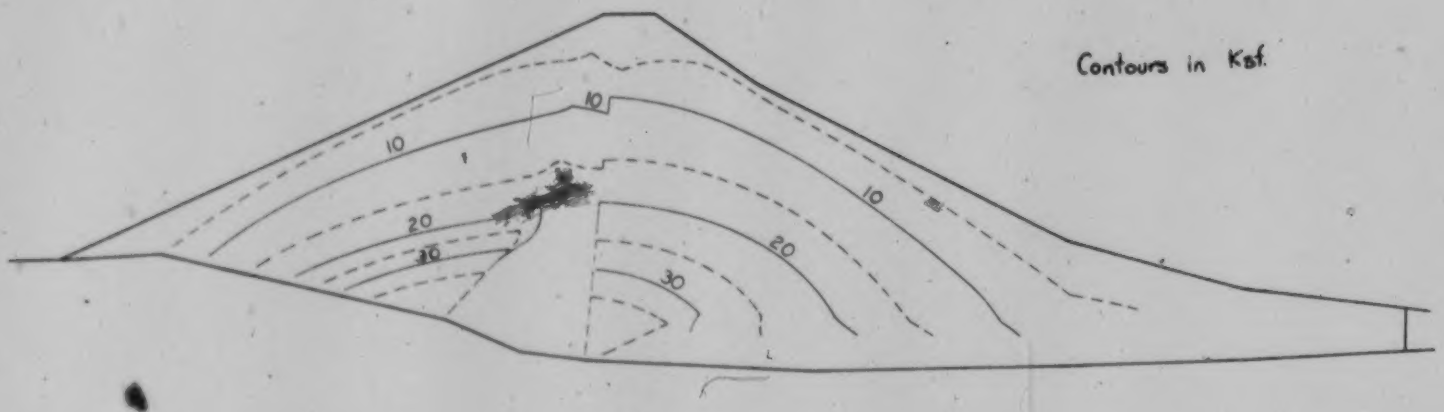
Contoors in Ksf.

TRANSVERSE SECTION: INTERMEDIATE PRINCIPAL STRESS; 3D, MULTILINEAR


10
10
10
20
20
20
30
30
30
(15)

Contours in Ksf.

TRANSVERSE SECTION: INTERMEDIATE PRINCIPAL STRESS; 3D, LINEAR+BEDROCK


Contours in Ksf.
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10
10
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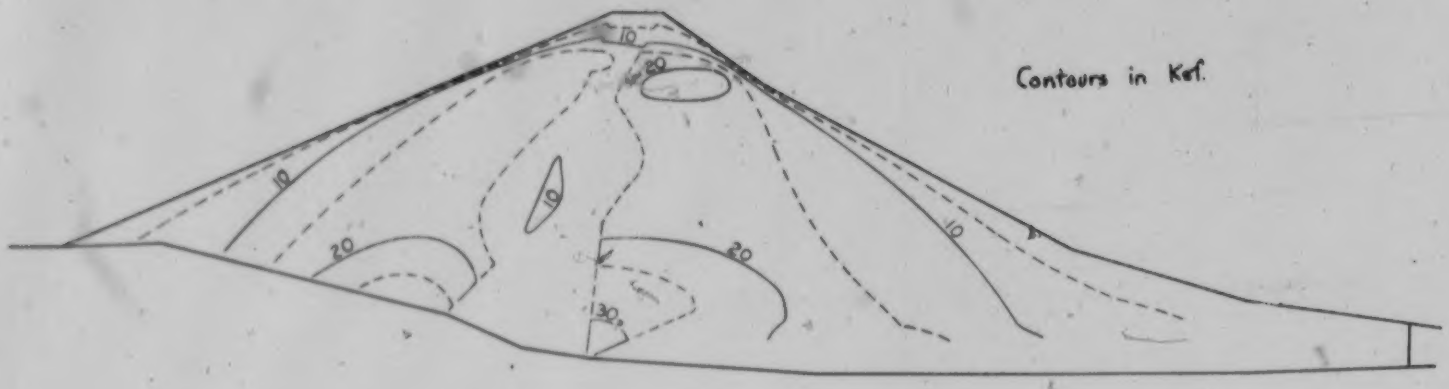
10
20
30

TRANSVERSE SECTION: INTERMEDIATE PRINCIPAL STRESS; 2D, PLANE STRAIN


Contours in Kef.
10
20
10
10
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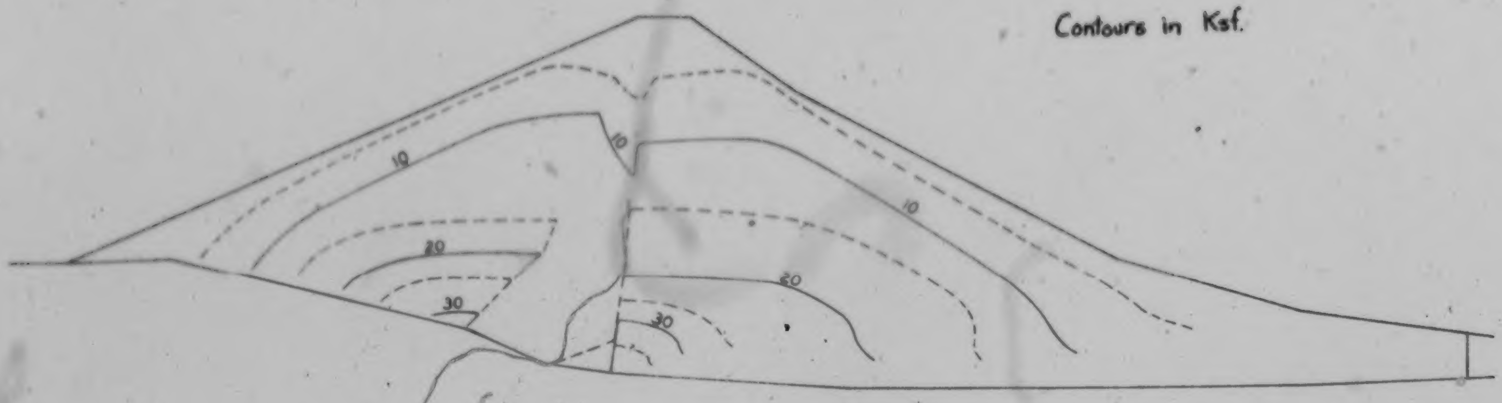
30
10

TRANSVERSE SECTION: INTERMEDIATE PRINCIPAL STRESS; 3D, LINEAR, 1 LIFT


Contours in Ksf.
10
10
10
20
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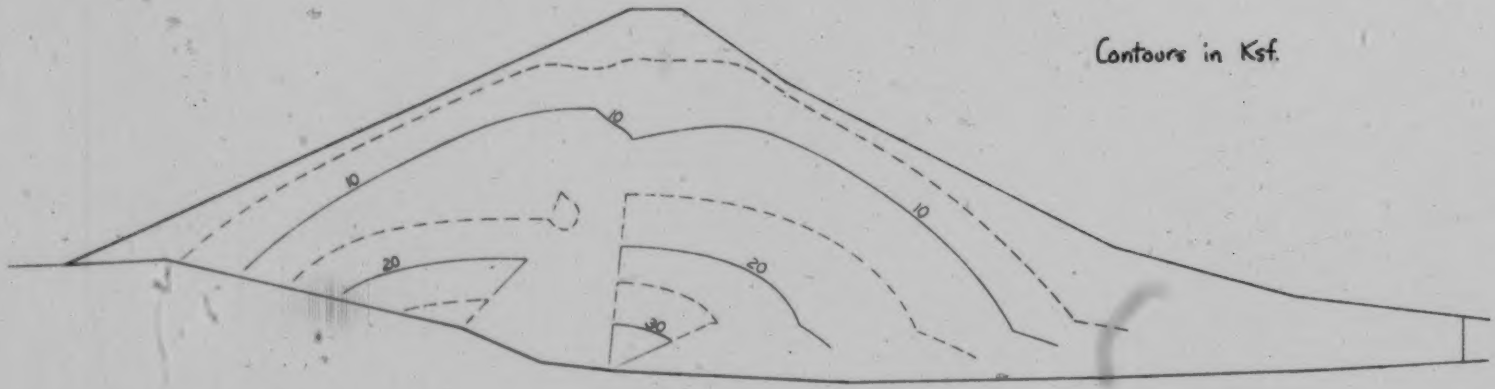
30
30

TRANSVERSE SECTION: MINOR PRINCIPAL STRESS; 3D, MULTILINEAR


10
10
10
20
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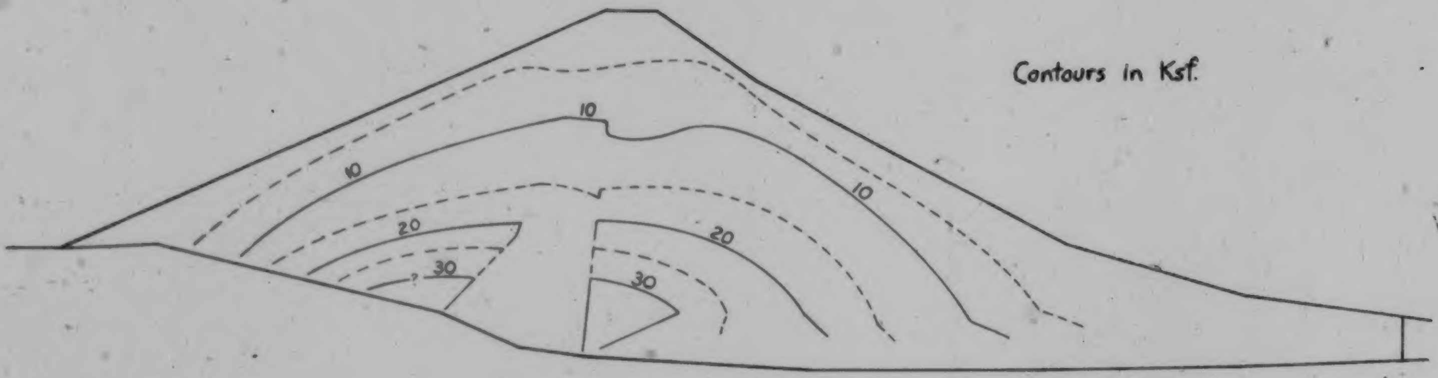
Contours in Ksf.

TRANSVERSE SECTION: MINOR PRINCIPAL STRESS; 3D, LINEAR+BEDROCK


Contours in Ksf.
10
10
10
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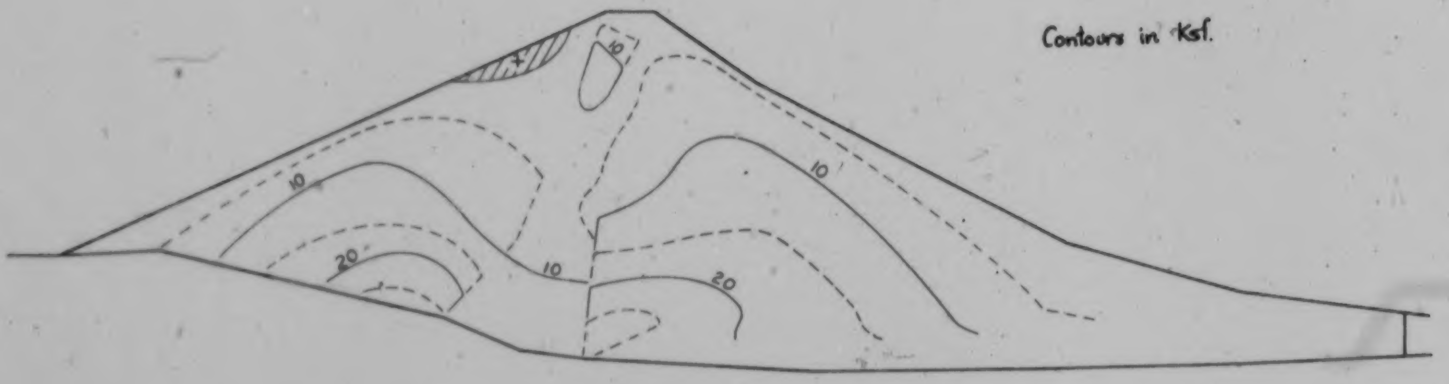
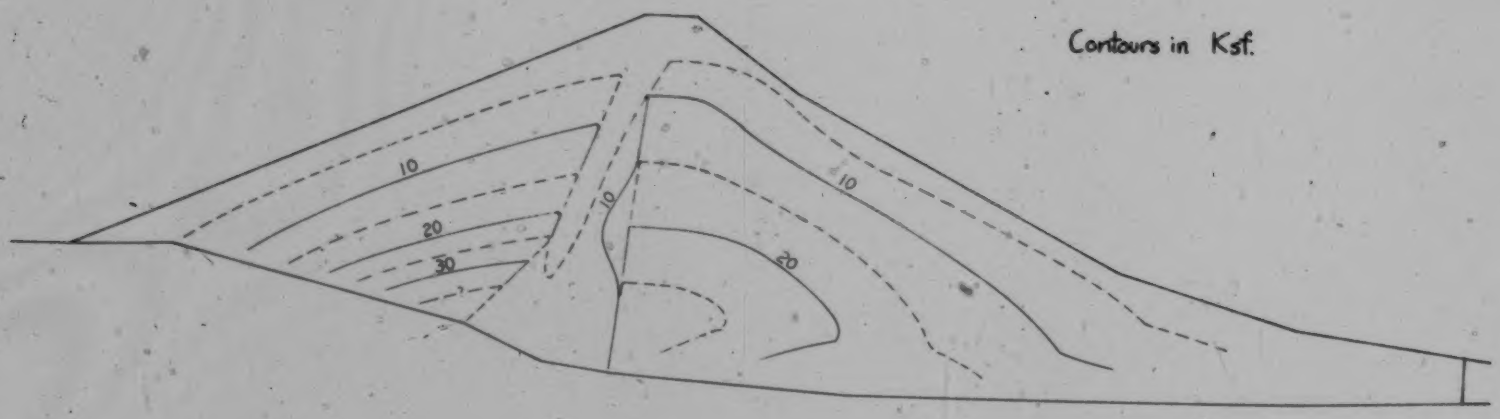
30

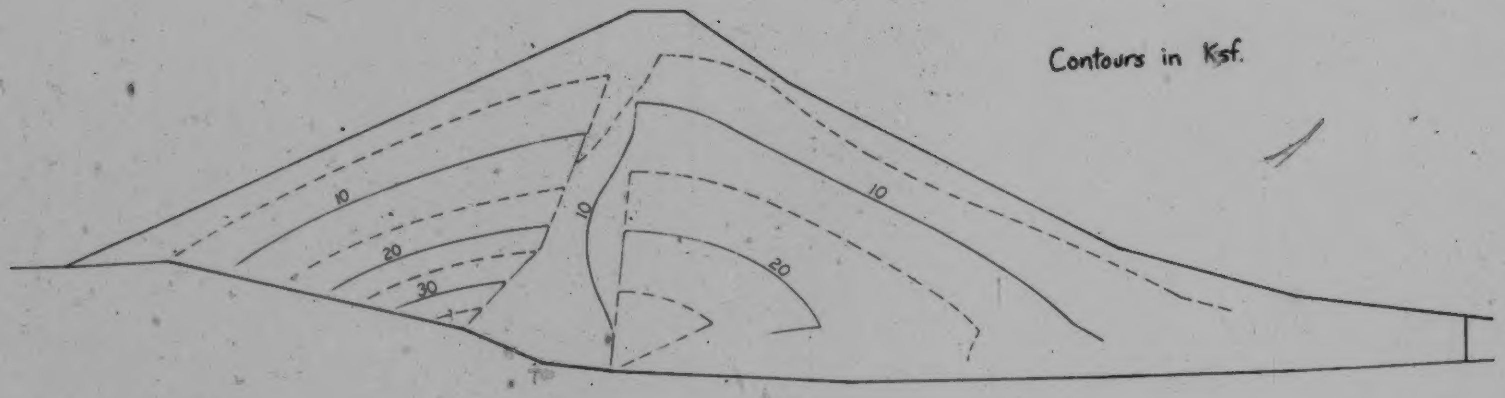
TRANSVERSE SECTION: MINOR PRINCIPAL STRESS; 2D, PLANE STRAIN



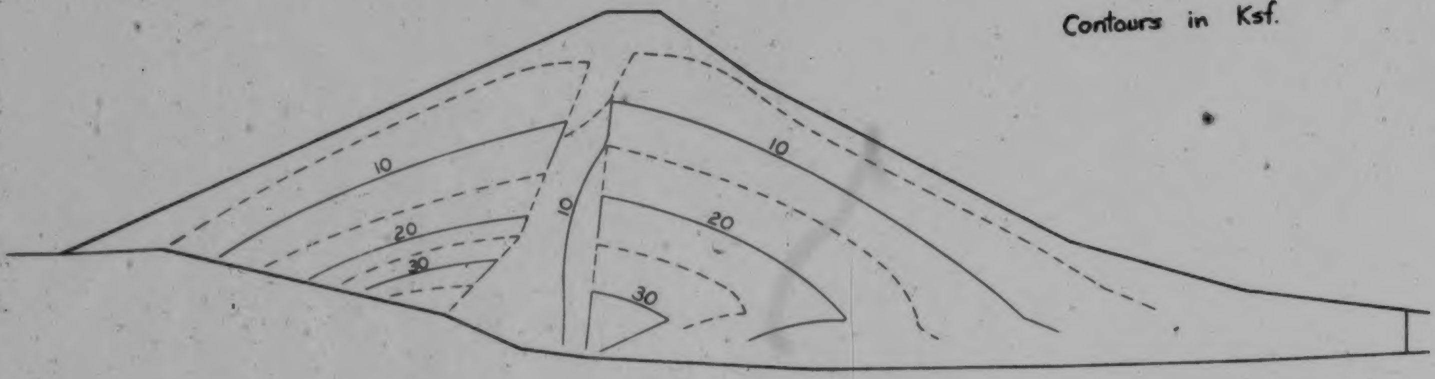
TRANSVERSE SECTION; MINOR PRINCIPAL STRESS; 3D, LINEAR, 1 LIFT


Contours in Ksf.
10
20
30
10
10
20

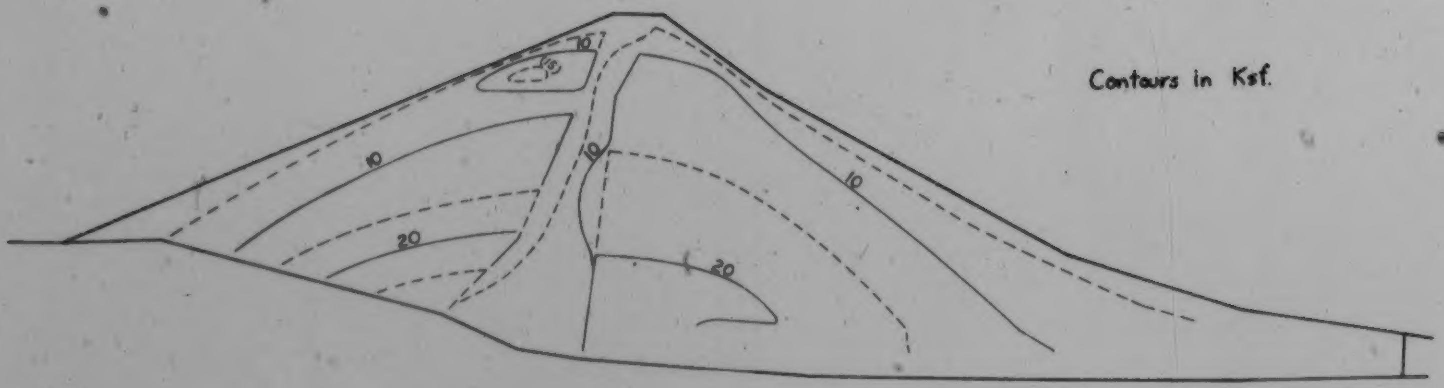
TRANSVERSE SECTION: MAXIMUM SHEAR STRESS; 3D, MULTILINEAR



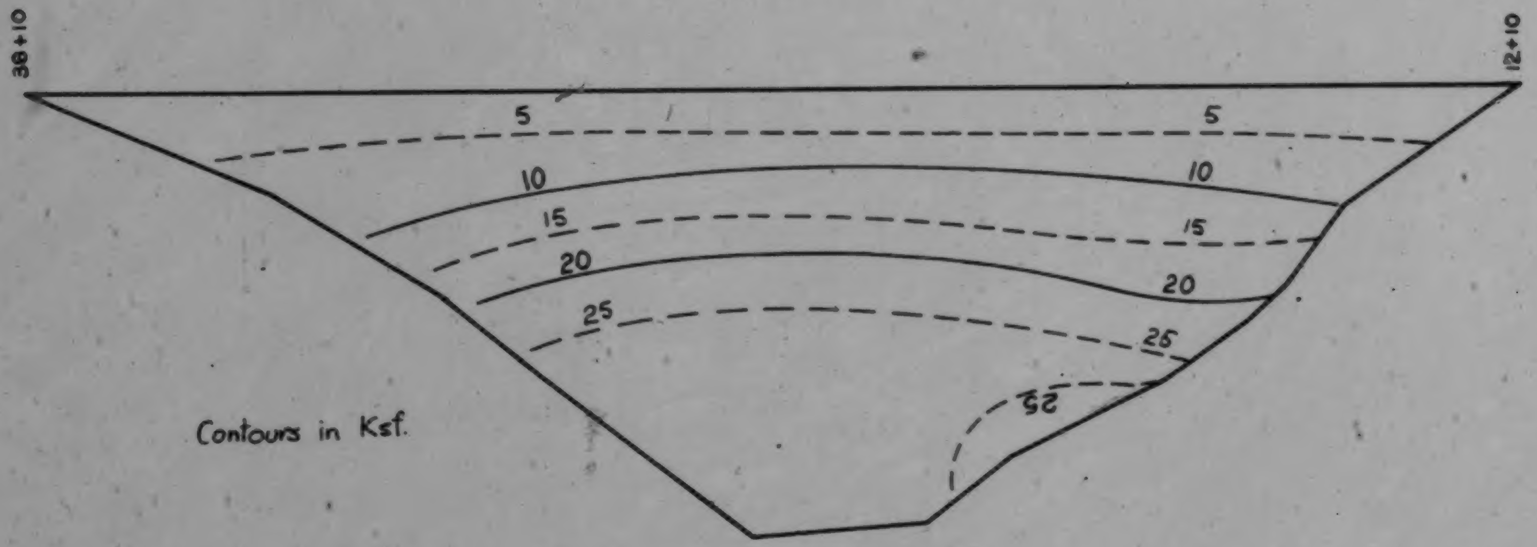
TRANSVERSE SECTION: MAXIMUM SHEAR STRESS; 3D, LINEAR+BEDROCK



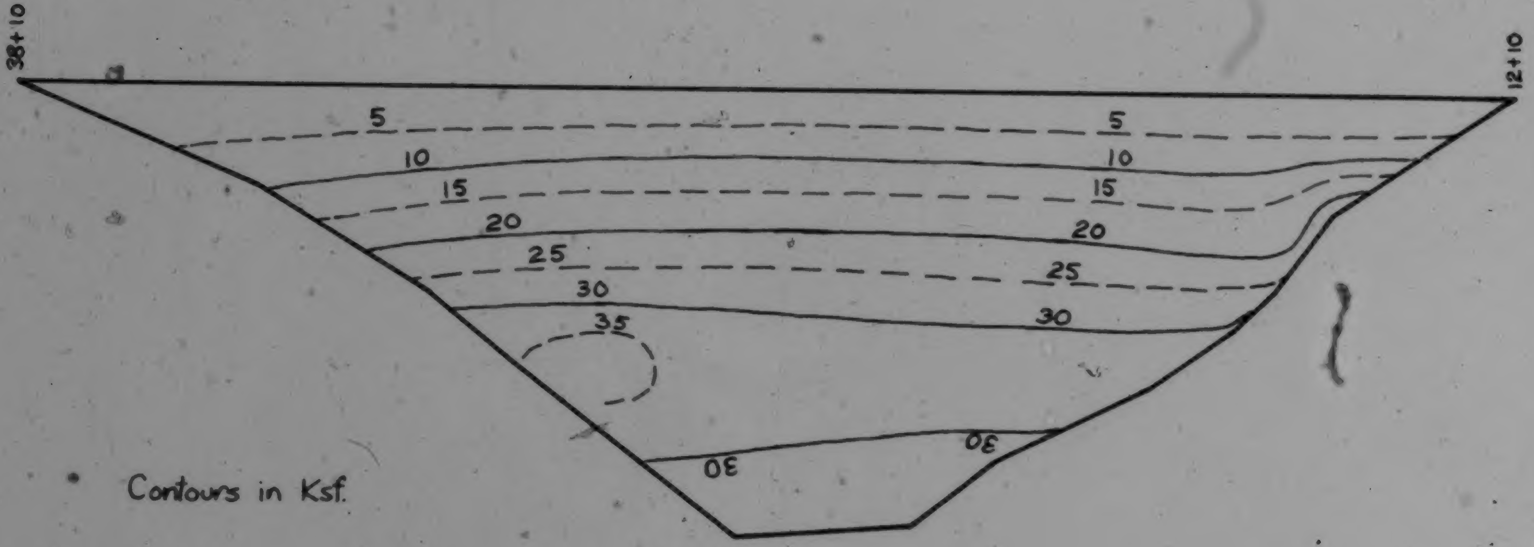
TRANSVERSE SECTION: MAXIMUM SHEAR STRESS; 2D, PLANE STRAIN


Contours in Ksf.
10
15
10
20
10
10
20

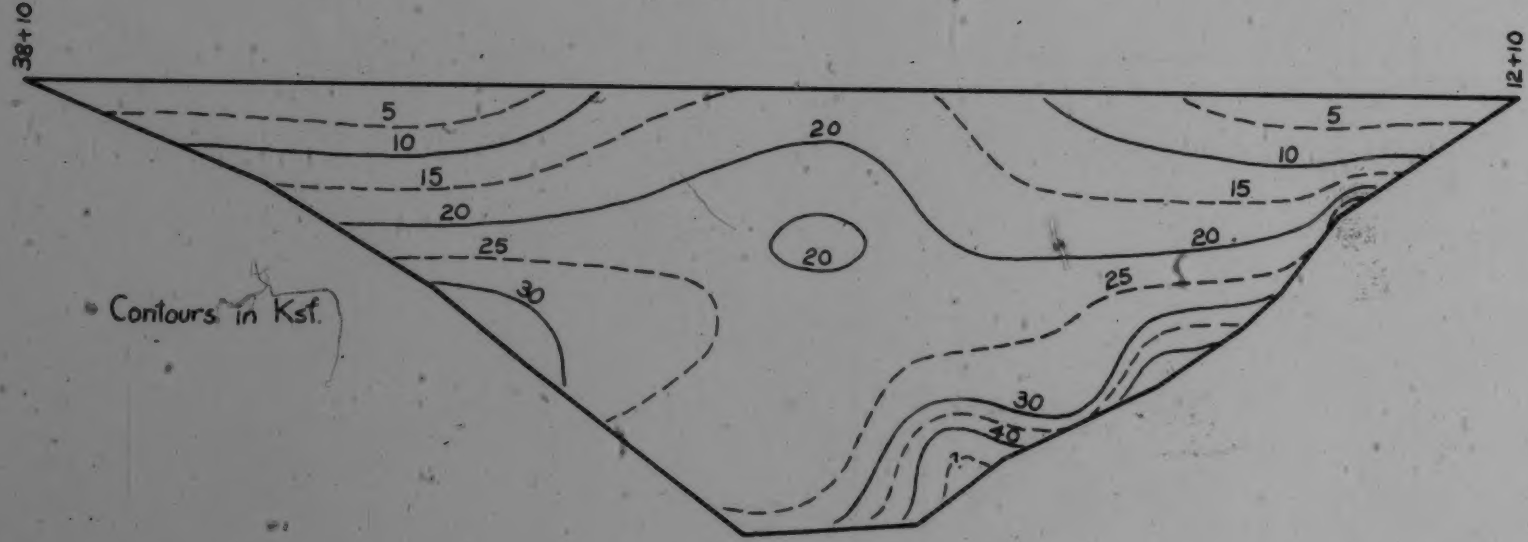
TRANSVERSE SECTION: MAXIMUM SHEAR STRESS; 3D, LINEAR, 1 LIFT


38+10
12+10
5
5
10
10
15
15
20
20
25
25
25
Contours in Ksf.

LONGIT. CORE SECTION: MAJOR PRINCIPAL STRESS; 3D, MULTILINEAR


38+10
12+10
5
5
10
10
15
15
20
20
25
25
30
30
35
30
30
Contours in Ksf.

LONGIT. CORE SECTION: MAJOR PRINCIPAL STRESS; 3D, LINEAR+BEDROCK


38+10
12+10
5
10
15
20
25
30
20
20
5
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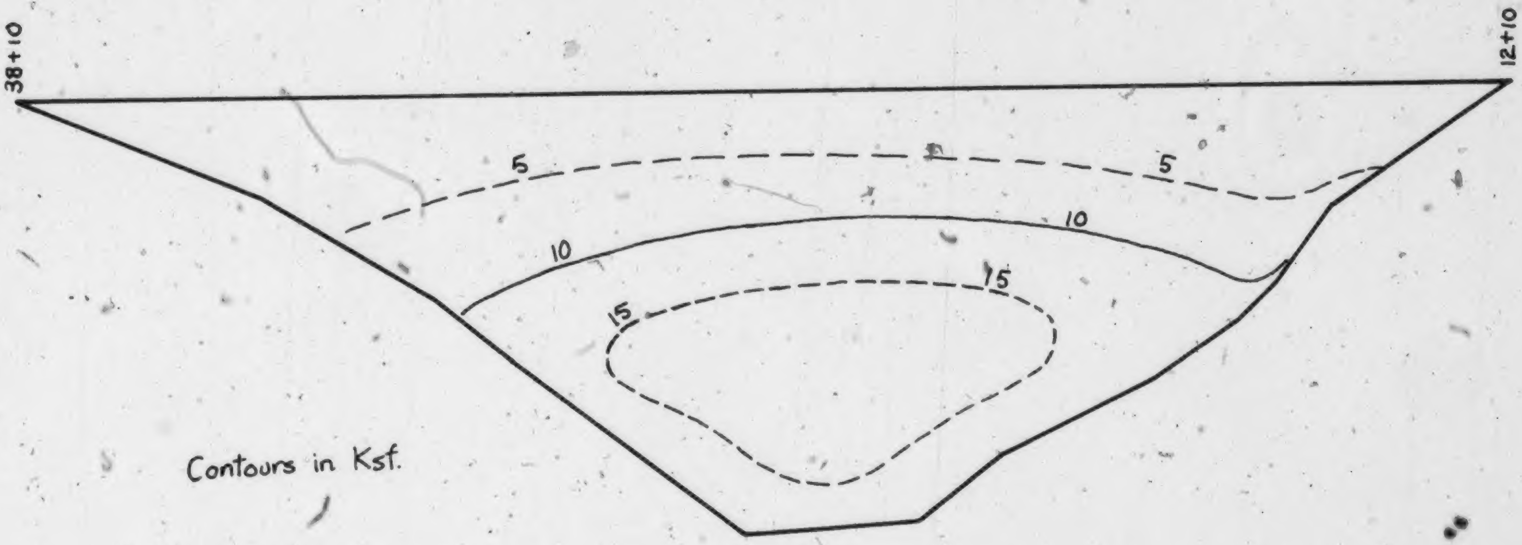
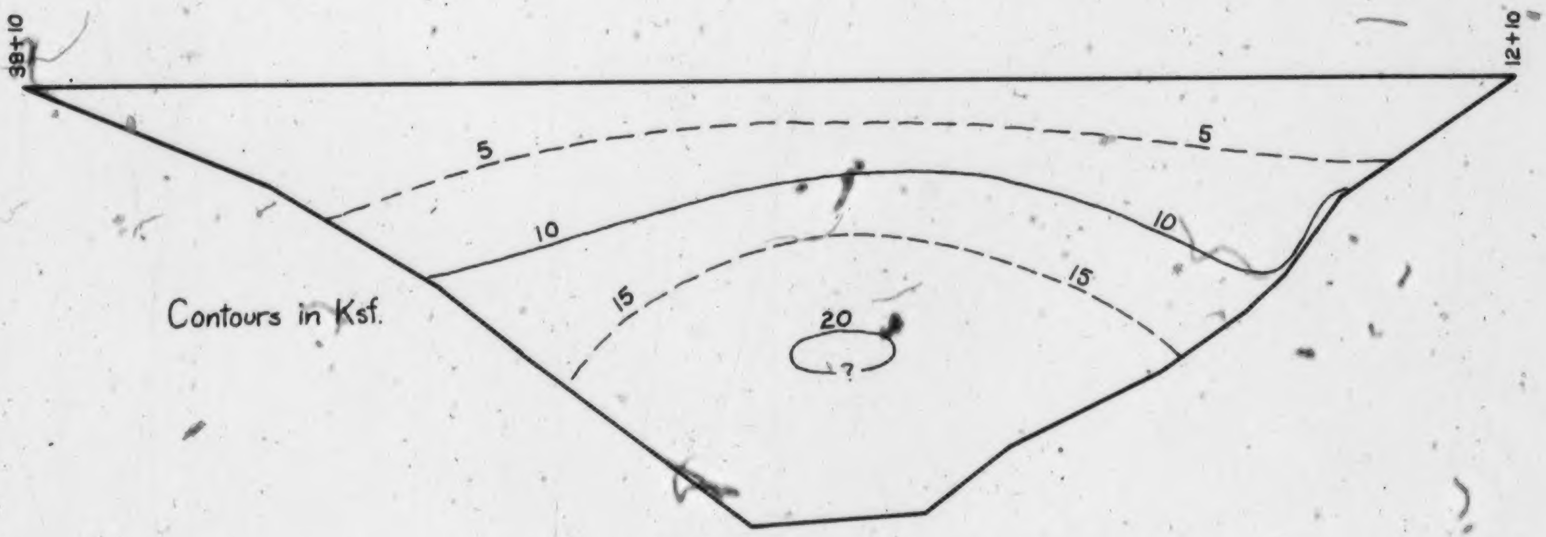
15
20
25
30
40
Contours in Ksf.

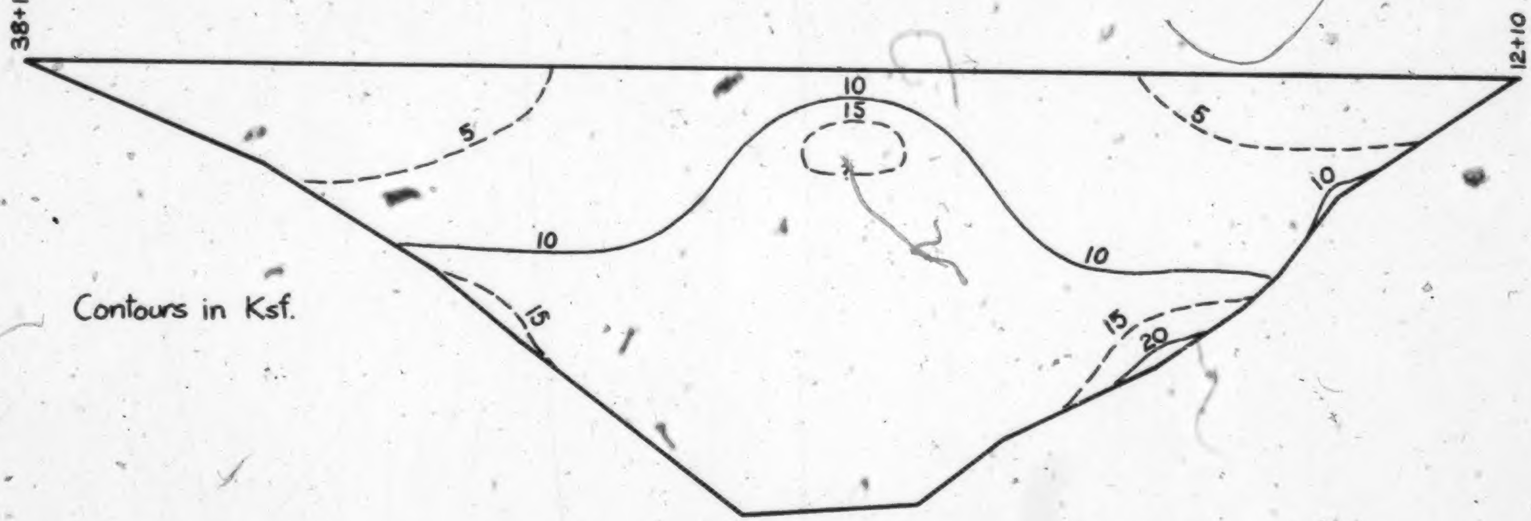
LONGIT. CORE SECTION: MAJOR PRINCIPAL STRESS; 3D, LINEAR, 1 LIFT


38+10
12+10
5
5
10
10
15
15
Contours in Ksf.

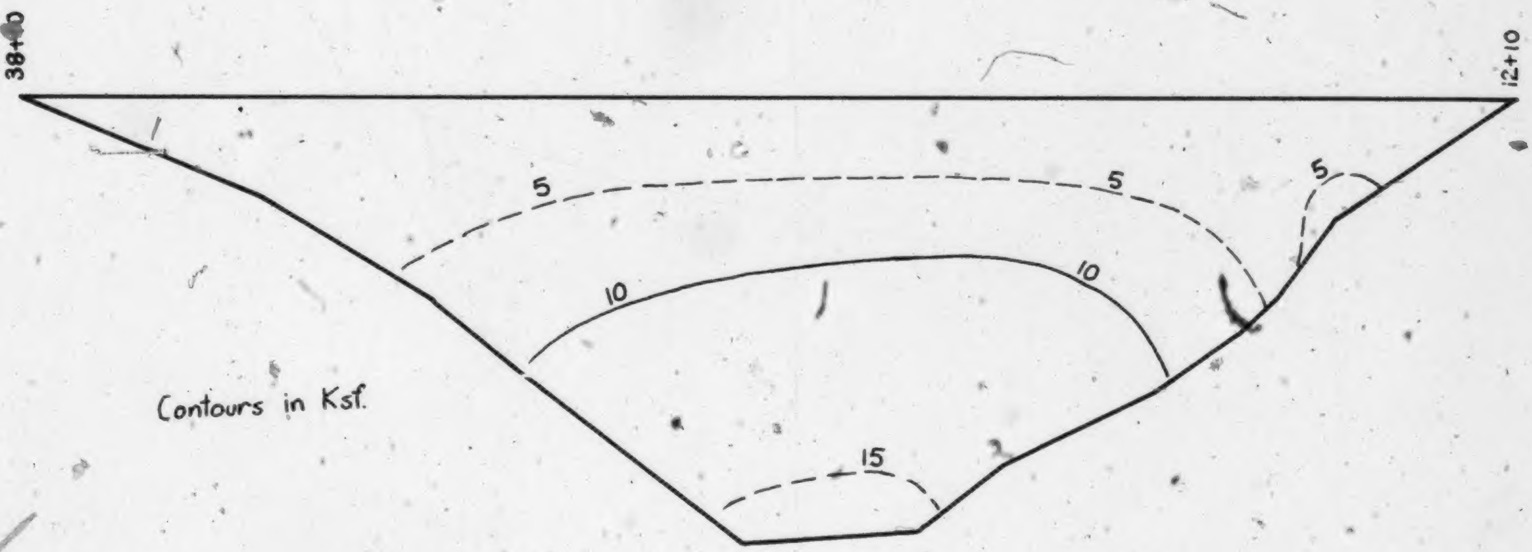
LONGIT. CORE SECTION: INTERMEDIATE PRINCIPAL STRESS; 3D, MULTILINEAR


38+10
12+10
5
5
10
10
15
15
20
?
Contours in Ksf.

LONGIT. CORE SECTION: INTERMEDIATE PRINCIPAL STRESS; 3D, LINEAR+BEDROCK

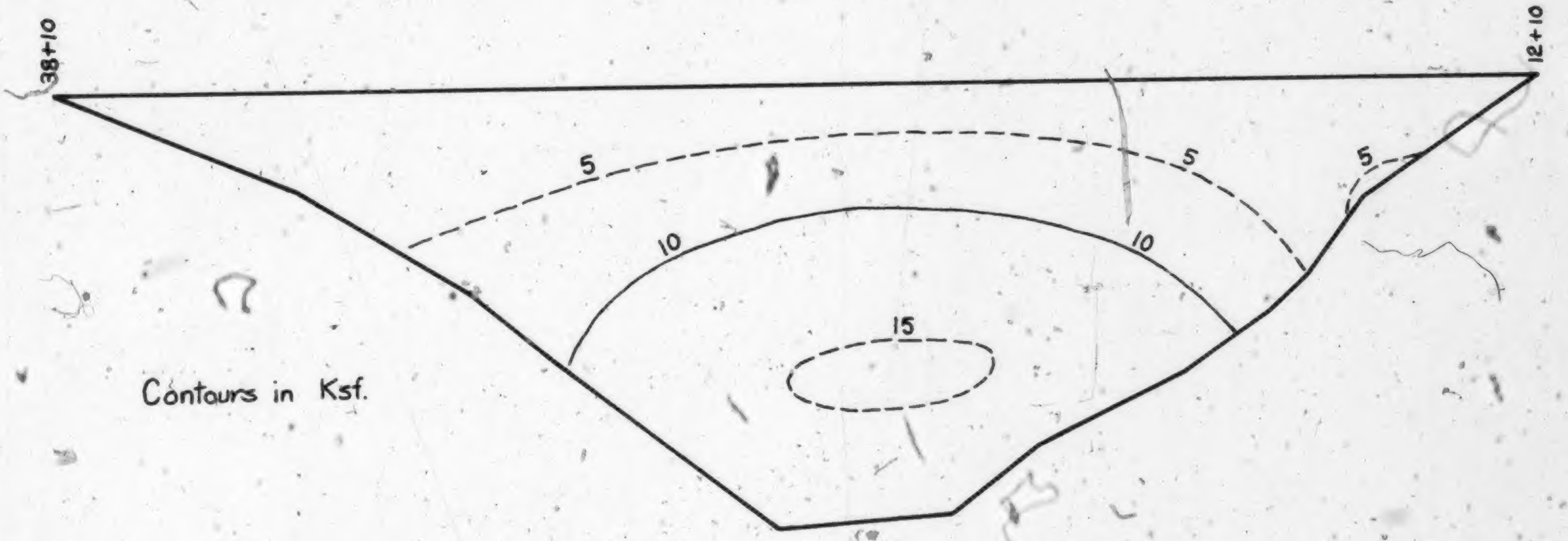

38+10
12+10
5
5
10
15
10
10
10
15
15
20
Contours in Ksf.

LONGIT. CORE SECTION. INTERMEDIATE PRINCIPAL STRESS; 3D, LINEAR, 1 LIFT

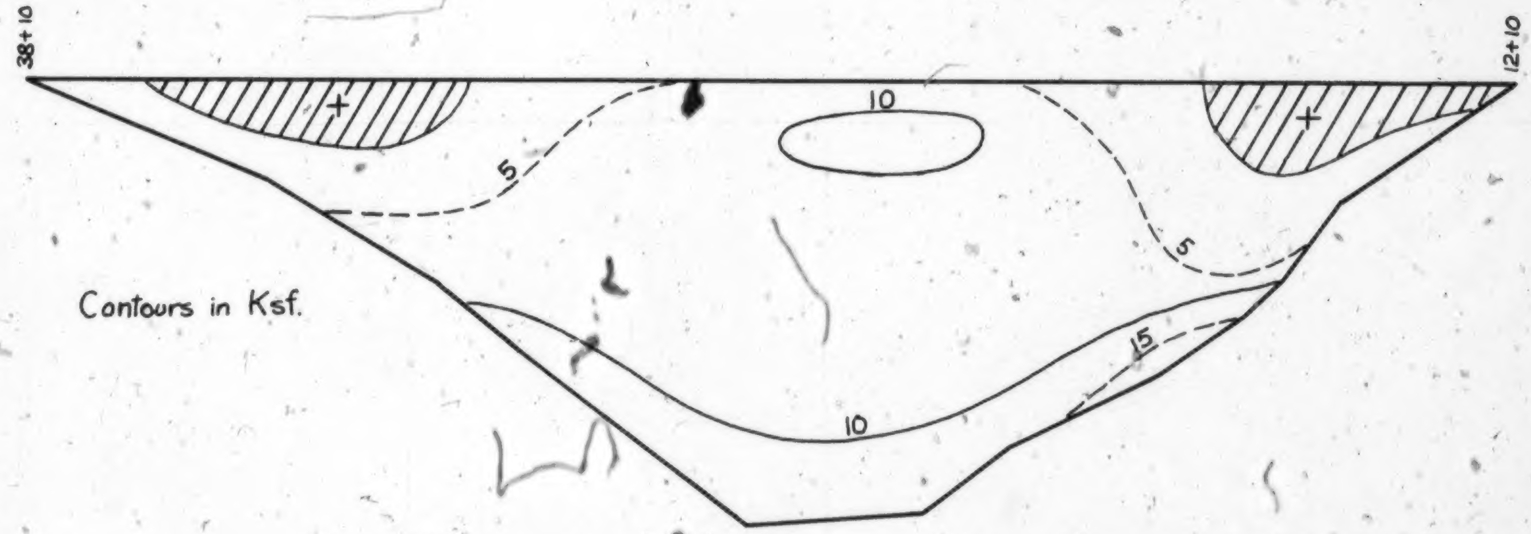

38+[illegible]0
12+10
1
5
5
5
10
10
15

Contours in Ksf.

LONGIT. CORE SECTION: MINOR PRINCIPAL STRESS; 3D, MULTILINEAR

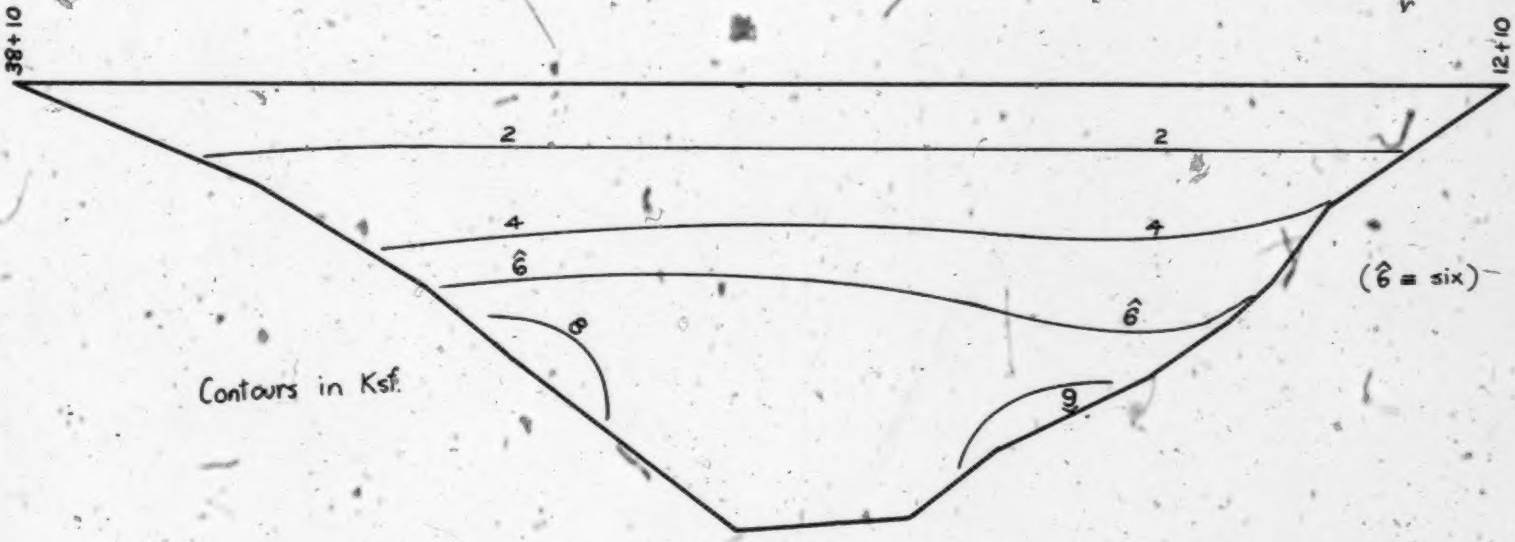

38+10
12+10
5
5
5
10
10
15
Contours in Ksf.

LONGIT. CORE SECTION: MINOR PRINCIPAL STRESS; 3D, LINEAR+BEDROCK

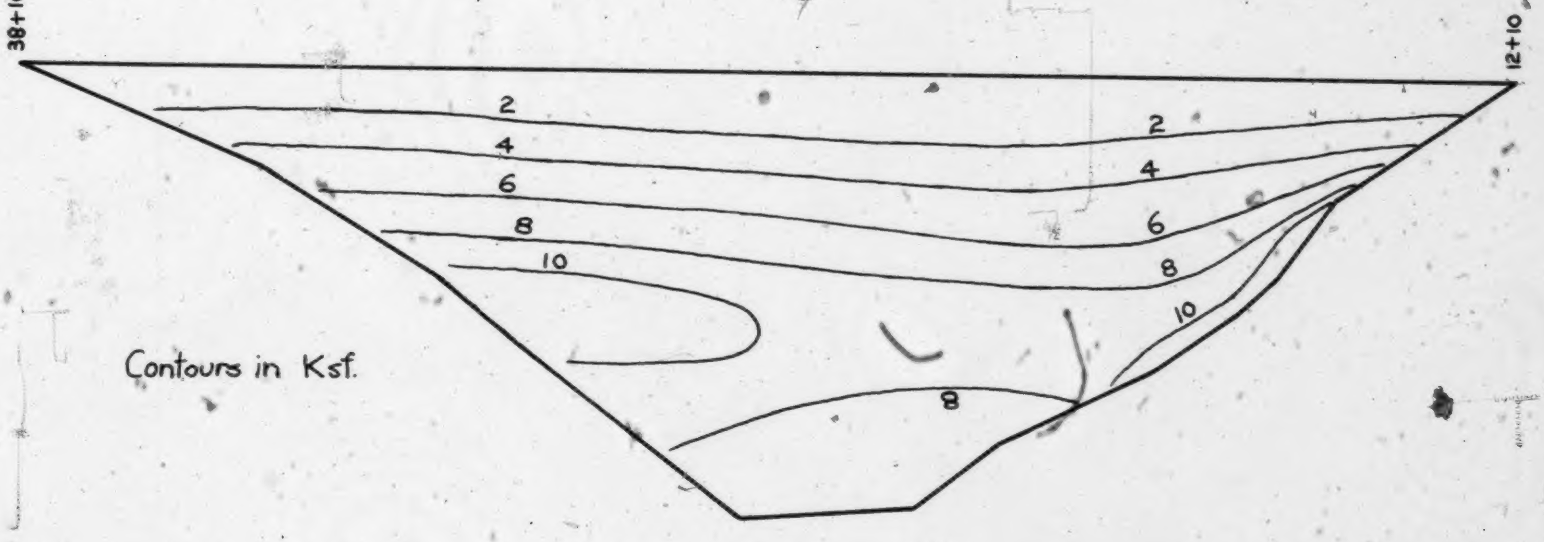

38+10
12+10
+
+
5
5
10
10
15

Contours in Ksf.

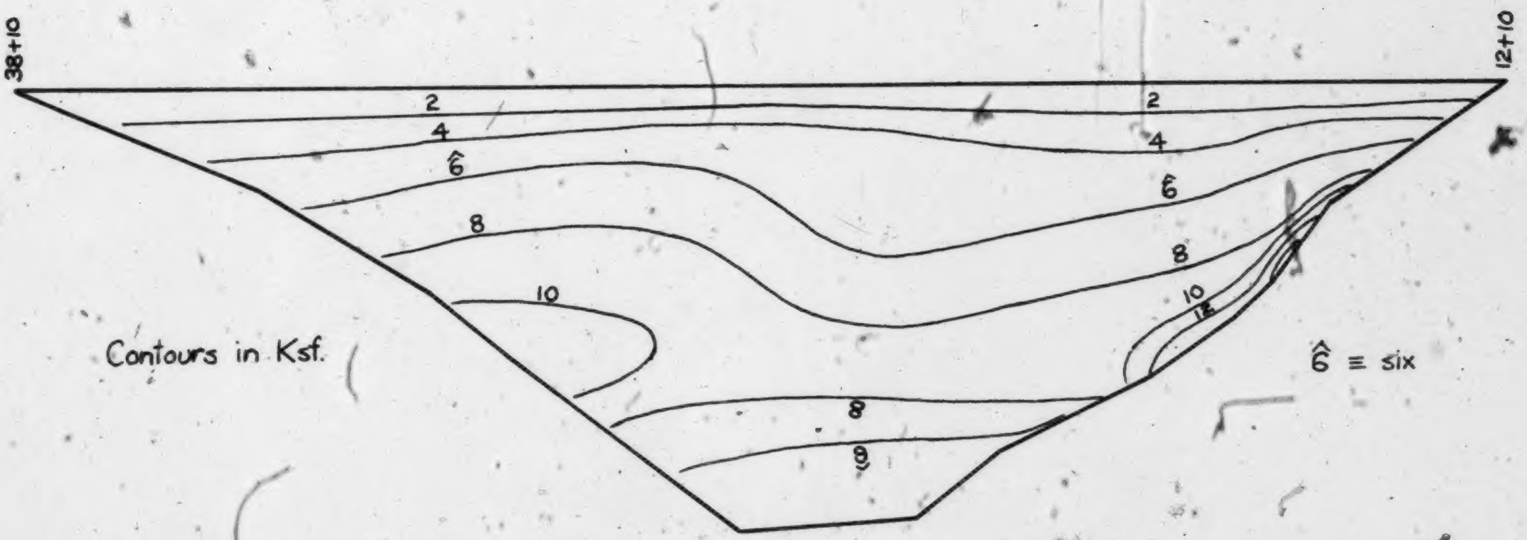
LONGIT. CORE SECTION: MINOR PRINCIPAL STRESS; 3D, LINEAR, 1 LIFT


38+10
12+10
2
2
4
4
$\hat{6}$
$\hat{6}$
($\hat{6}$ = six)
8
9
Contours in Ksf.

LONGIT. CORE SECTION: MAXIMUM SHEAR STRESS; 3D, MULTILINEAR



LONGIT. CORE SECTION: MAXIMUM SHEAR STRESS; 3D, LINEAR+BEDROCK



Contours in Ksf.

$\hat{6} \equiv$ six

LONGIT. CORE SECTION: MAXIMUM SHEAR STRESS; 3D, LINEAR, 1 LIFT

APPENDIX E

COSTS OF THE FENA.3D ANALYSES

Apart from the previously mentioned data preparation costs, the chief costs in computation are the generation of structure stiffness and the solution of the large set of simultaneous linear equations to obtain nodal displacements. Structure stiffness is composed piecewise of element stiffness, thus the cost of this factor is proportional to element type, integration procedure, and the total number of elements treated. The solution time of the equations depends upon the semi-bandwidth of the set, which is a property of the mesh of elements. Other costs are marginal, and for evaluation purposes the cost of an analysis is a function of total elements generated and equation semi-bandwidth.

Krishnayya (1973b) presented a correlation of semi-bandwidth and CPU time for solution of the equation set. This is shown in Fig. E.1. It must be remembered that, as used, the program FENA.3D depends heavily for its economy upon MTS system subroutines set up for the IBM 360/67 at the University of Alberta. The writer found a more useful prediction was the total CPU time for a run, correlated with the product of semi-bandwidth and total elements. The relative cost of CPU storage increases with the overall core area required for a problem too. Some costs of various runs for the Mica Dam analyses are given in Table E.1.

Finally, an information sheet conforming to the requirements for the W. E. S. Symposium (Desai, 1972) is given.

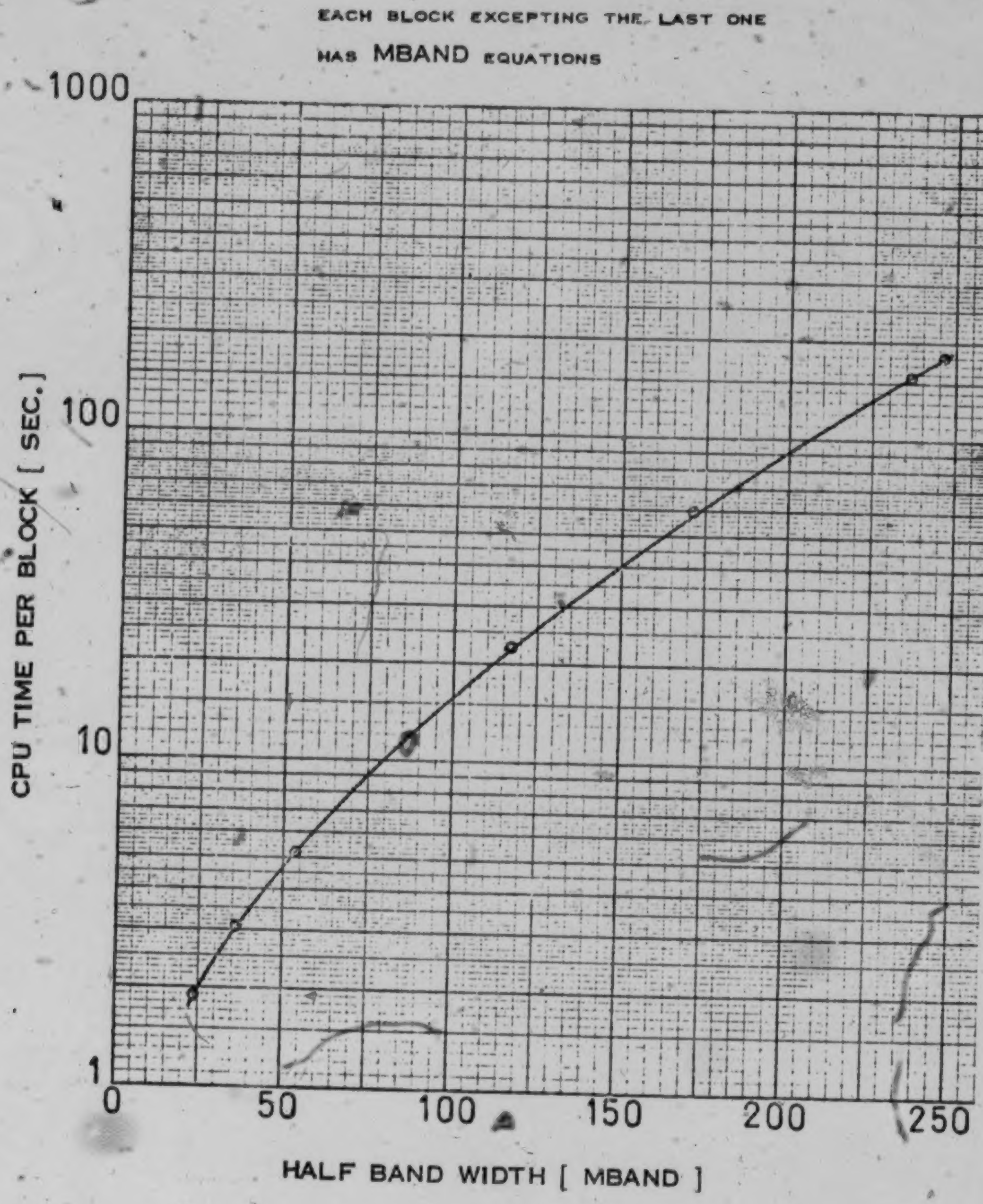


FIGURE E.1 APPROXIMATE CPU TIME FOR SOLUTION OF EQUATIONS IN PROGRAM FENA.3D (after Krishnayya, 1973b)

TABLE E.1

RUNNING STATISTICS, PROGRAM FENA.3D

2 2-point integration 3 3-point integration 1 1972-73 rates 4 1973-74 rates

Run	Element Total	Equations	MBAND	Elapsed (min)	CPU (secs)	CPU store (page-min)	CPU time	CPU store	Total
							Cost ($Canadian)		
1:1 dam, 4 lifts	20[2]	105	48	16	97	109	2[1]	1.5	3.8
1:1 dam, 8 lifts	540[2]	495	138	75	1154	1973	25[1]	26	52
1 step, linear	254[3]	828	183	141	1230	2905	40[4]	57	98
5 step, linear	708[3]	828	183	206	3339	8117	109[4]	159	277
5 step, multilinear	1416[3]	828	183	390	6810	15980	223[4]	318	565
5 step, lin., 2D pl. strn.	135[3]	270	177	62	663	1532	21[4]	30	55

COMPUTER CODE (PROGRAM) QUESTIONAIRE

1. Name of Code, if any: FENA.3D
2. Purpose of the Code: To compute stresses and strains using linear or non-linear stress-strain relationships.
3. Developed by (including year of development): Krishnayya, A.V.G.(1973)
4. Modified by or revised by (including year of modification or revision): Simmons (1974) for multilinear analysis and tapewriting options.
5. Code available from: Dept. Civil Eng., Univ. of Alberta, Edmonton.
6. Brief description of the problems solved by using the Code: Three dimensional constructional and cracking analysis of earth dams; Building settlements, Excavation movements.
7. Other types of problems for which suitable: Any three dimensional problem with linear or non-linear stress-strain relationships.
8. Type of finite element used: Isoparametric hexahedron with 8 nodes.
   Type of nonlinear technique used: Piecewise incremental procedure.
   Type of displacement function (or field variable function) used (e.g. linear, quadratic, etc.): Linear.
   Type of equation solver used: Gaussian reduction and back-substitution in blocks.
9. Language used: FORTRAN IV .
   Machine on which presently adopted: IBM 360/67 with MTS operating system.
   Capacity in terms of number of nodes and/or elements: Maximum elements 350, Maximum nodes 450, Maximum semi-bandwidth 250.
   Is documentation such as Guide for Data Input, etc. available? Yes.
10. Other comments such as number of cards in the code, computer time required for a typical problem, limitations if any: 1545 cards. See Table E.1 for guide to times. Uses 8 MTS system subroutines to achieve economy. Requires two temporary disk files, optionally also one tape-drive. Data preparation time and checking procedures may be measured in months.